Coastal Storm Damage Reduction Study
Reconnaissance Report

BLACK ROCK & SHORT BEACHES NAHANT, MASSACHUSETTS

March 1995

US Army Corps
of Engineers
New England Division

SECTION 103
RECONNAISSANCE REPORT

BLACK ROCK AND SHORT BEACHES, NAHANT, MASSACHUSETTS
COASTAL STORM DAMAGE REDUCTION STUDY

MARCH 1995
U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY (Leave blank)

2. REPORT DATE
February 1995

3. REPORT TYPE AND DATES COVERED
Final

4. TITLE AND SUBTITLE
Black Rock and Short Beaches, Nahant, Massachusetts
Coastal Storm Damage Reduction Study
Reconnaissance Report

5. FUNDING NUMBERS

6. AUTHOR(S)

Charles L. Joyce and Study Team

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers
New England Division
424 Trapelo Road
Waltham, Massachusetts 02254-9149

8. PERFORMING ORGANIZATION REPORT NUMBER

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers
New England Division
424 Trapelo Road
Waltham, Massachusetts 02254-9149

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES

12a. DISTRIBUTION/AVAILABILITY STATEMENT

Approval for public release; distribution is unlimited

12b. DISTRIBUTION CODE

13. ABSTRACT (Maximum 200 words)

The coastal storm damage reduction reconnaissance level study was conducted by the New England Division, U.S. Army Corps of Engineers at the request of the Town of Nahant, Massachusetts for assistance in controlling coastal flooding in the backshore of Black Rock and Short Beaches, known as the Lowlands Area and located in the northwest part of the town. The study was funded under the Continuing Authority Program under Section 103 of the 1962 River and Harbors Act, as amended, for the purpose of damage reduction from coastal storms. The Reconnaissance Report documented the study that concluded that the formulated plans to control coastal flooding were not economically justified and that there was no opportunity for Corps of Engineers assistance for controlling flooding and reducing damages in the Study Area.

14. SUBJECT TERMS

Nahant, Massachusetts; Black Rock Beach; Section 103 Study; coastal flooding; damage reduction

15. NUMBER OF PAGES
91

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
Unclassified	Unclassified	Unclassified	

EXECUTIVE SUMMARY

This Reconnaissance Report was prepared by the New England Division, Corps of Engineers in response to a request from the town of Nahant, Massachusetts for assistance in solving the coastal flooding problem in the Lowlands Area located in the northwest part of the town. The study was conducted under the Continuing Authority of Section 103 of the 1962 River and Harbors Act, as amended, for the purpose of reducing flood damages from coastal storms. The purpose of the Reconnaissance Study was to analyze the problem of flooding, formulate and evaluate alternative measures to control flooding and to determine if further study was warranted.

The Study Area comprises the triangular shaped northwest corner of the town of Nahant. It extends southeast along Short Beach and southwest along Black Rock Beach and includes the backshore common to both beaches known as the Lowlands Area. See Figures 1 and 2. More frequent flooding of the backshore occurs as a result of the overtopping of Black Rock Beach and more rarely via Short Beach. The study was limited to the protection primarily of residential properties in the area located along Castle Road, Castle Way, Ward Road, Foxhill Road, and Harbor View Road and the Lowlands Park and Playground. Here 53 structures (50 residential structures and land assessed at approximately $7.6 million, one pharmacy, a sewage pumping station and an electrical sub-station) are located. All but 10 of the structures have first floor elevations above the 100-year recurrence flooding event.

Flooding of the residential and commercial properties along Nahant Road beginning west of Spring Road and extending east is caused by the overtopping of the riprap revetment in the Short Beach backshore. While the damages to these properties can be significant in severe storms, they are not sufficient to justify additional protection. They have therefore been excluded from the Study Area.

The Study Area is subject to flooding and ponding primarily as a result of tidal surges and limited direct wave action caused by coastal storms across Black Rock Beach. Here flooding occurs mainly due to the entry of storm waters through the Town Beach portion of Black Rock Beach and through the properties without seawalls. For more rare storm events, flood waters enter the Study Area from tidal surges and wave action across the riprap revetment at Short Beach.

A two step process was employed in formulating improvements for controlling flooding. First, those alternatives that alleviate flooding through Black Rock Beach were considered. Improvements across Short Beach would then be identified and evaluated if a feasible solution along Black Rock Beach were found with sufficient residual economic benefits.

An initial screening process to reduce flooding from Black Rock Beach eliminated an offshore breakwater, a revetment, a dike and non-structural alternatives for technical and/or economic reasons. Three alternative plans were evaluated in detail: 1) a 1100-foot long seawall across Black Rock Beach, 2) a system of partial seawalls across Town Beach and across those properties not having seawalls and tied into the existing seawalls and 3) a series of sandfills in the same areas as the partial seawall. These were designed to minimize overtopping of the beach to a negligible amount and would therefore reduce damages due to ocean flooding except when a 100-year recurrence event were exceeded. None of the three alternative plans were found to be economically feasible.

Further federal participation in Section 103 Authority studies to protect the Study Area is not warranted and Corps of Engineers involvement ends with the publication of this report.

The lack of adequate drainage increases the depth and extends the duration of ponding in the Black Rock and Short Beaches backshore. The solution to the interior drainage problem, which is not within the purview of the Corps of Engineers, has two possible components; one discharges into Nahant Harbor and the other would direct storm water flow into Lynn Harbor. The principal drainage system for the area starts with a ditch constructed parallel to Ward Road, succeeded by series of culverts and pipes leading to double 24 inch pipe outfall with tidal gate at Pond Beach which drains into Nahant Harbor. Another shorter more direct drainage system could be developed under Black Rock Beach into Lynn Harbor.

Continued improvement of procedures for providing residents of the flood prone area with as much advanced flood warning as possible in order to reduce the risk of injury to and loss of human life is desirable. Property owners would be enabled to temporarily raise property above and/or remove it from the flooded area. Given the loss of vehicles during former storms, their removal prior to storm events would seem, in many cases, to be an avoidable loss.

TABLE OF CONTENTS

I MAIN REPORT

TABLE OF CONTENTS (Continued)

I MAIN REPORT

I MAIN REPORT

Chapter 1: INTRODUCTION

STUDY AREA

The Black Rock and Short Beaches Shore Protection Study Area lies in the town of Nahant, Essex County, Massachusetts some eight miles northeast of Boston Harbor. Short Beach is referenced on some maps as Little Nahant Beach. The Study Area comprises the triangular shaped northwest corner of the town of Nahant. It extends southeast along Short Beach and southwest along Black Rock Beach and includes the backshore common to both beaches known as the Lowlands Area. See Figures 1 and 2. More frequent flooding of the backshore occurs as a result of the overtopping of Black Rock Beach and more rarely via Short Beach. This study is limited to the protection of the properties in the area located along Castle Road, Castle Way, Ward Road, Foxhill Road, and Harbor View Road and the Lowlands Park and Playground.

Flooding of the commercial and residential properties along Nahant Road beginning west of Spring Road and extending east is caused by the overtopping of the riprap revetment in the Short Beach backshore. While the damages to these properties can be significant in severe storms, they are not sufficient to justify additional protection. They have therefore been excluded from the study area.

STUDY AUTHORITY

This report has been prepared under the Continuing Authority of Section 103 of the 1962 River and Harbors Act, as amended, for the purposes of shore protection and flood damage reduction from coastal storms. The Section 103 Authority authorizes the Chief of Engineers of the U.S. Army Corps of Engineers to conduct pre-authorization studies for shore protection projects and, if appropriate, to construct them.

STUDY OBJECTIVE AND SCOPE

The purpose of this reconnaissance study is to determine whether further planning to reduce storm damages in the Black Rock and Short Beaches Study Area is justified from the point of view of the Federal Government.

PRIOR STUDIES AND REPORTS

There are no prior studies that address storm induced damages including flooding in the Black Rock and Short Beaches Study Area in Nahant, Massachusetts.

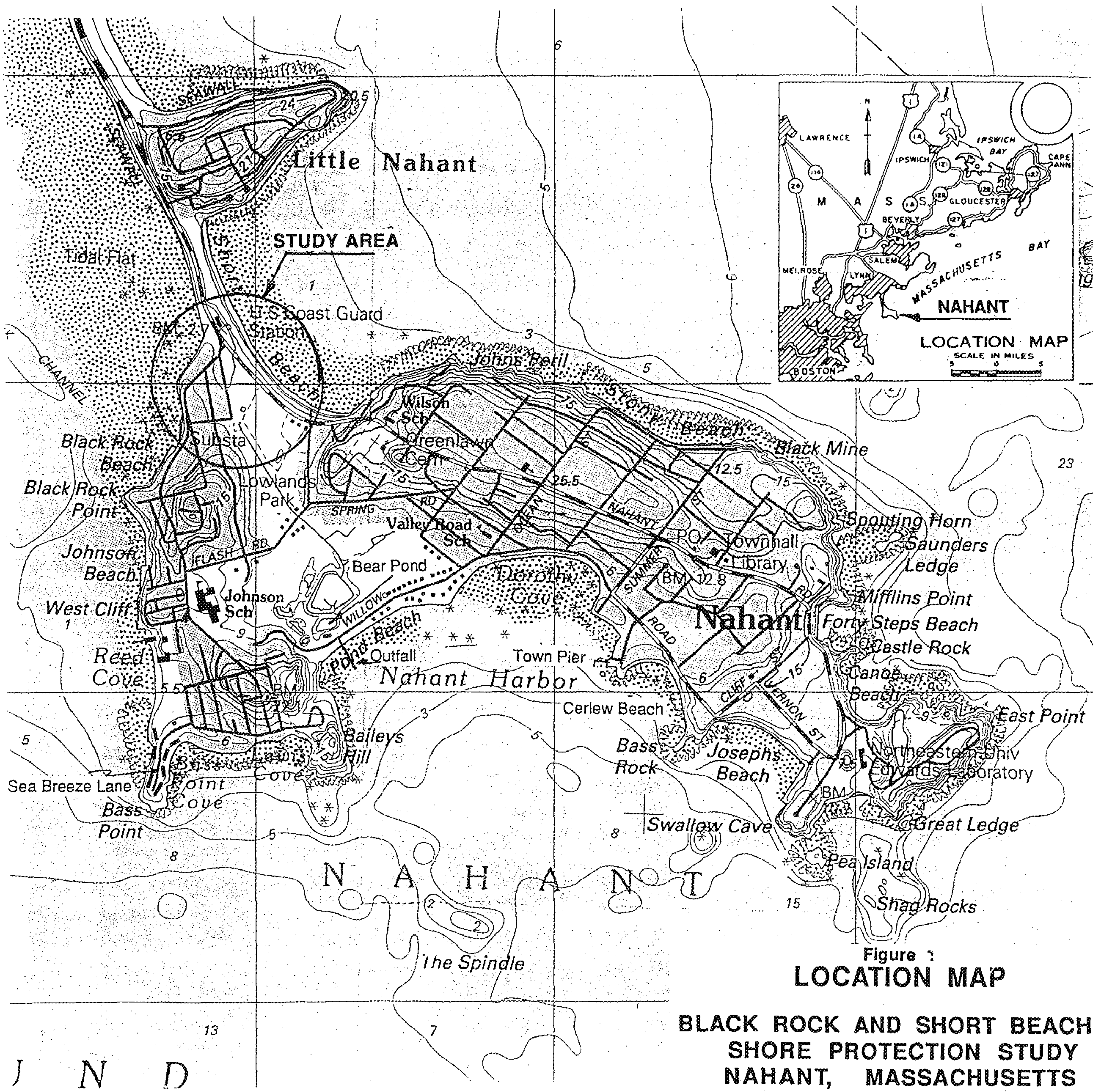


Figure 1
LOCATION MAP

BLACK ROCK AND SHORT BEACHES SHORE PROTECTION STUDY NAHANT, MASSACHUSETTS

Scale 1: 17,000 Contour Interval: 3 Meters
SOURCE: U.S.Geological Survey Map - 1: 25,000
Lynn, Massachusetts - Edited 1985

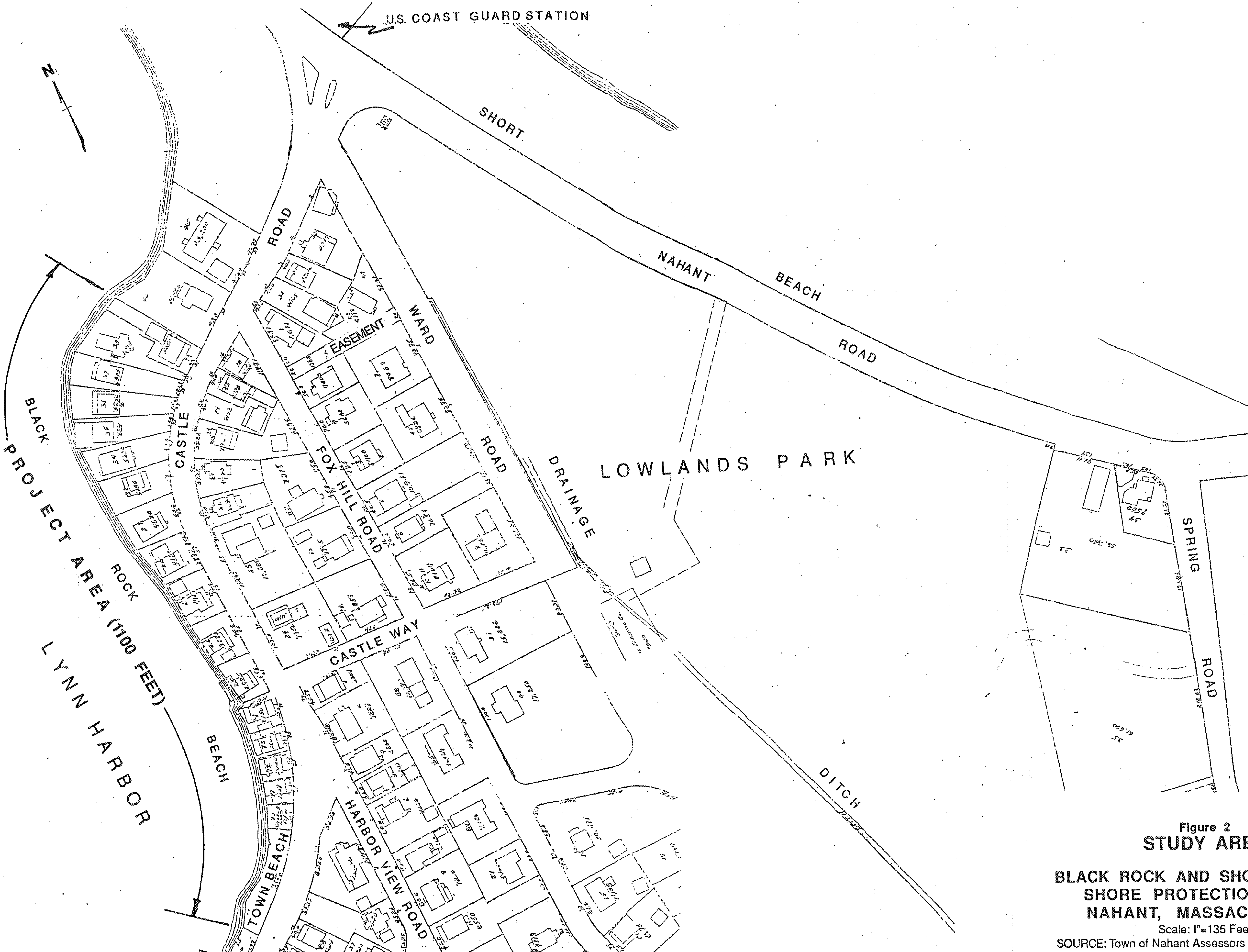


Figure 2
STUDY AREA

BLACK ROCK AND SHORT BEACHES
SHORE PROTECTION STUDY
NAHANT, MASSACHUSETTS

Scale: I"=135 Feet
SOURCE: Town of Nahant Assessors Maps - Revised 1993

Chapter 2: Existing Conditions

PHYSICAL SETTING

Black Rock Beach

The Study Area consists of approximately 1100 feet of Black Rock Beach shoreline extending north from the southern limit of Town Beach to the shore fronting the residence at 45 Castle Road. This portion of Black Rock Beach is subject to flooding during coastal storms. The shoreline is lined with single family and mainly year round homes which front on Castle Road, except for the southern portion which is occupied by the 150 foot long Town Beach with a crest elevation of 9.8 feet NGVD. Privately owned seawalls ranging in top elevations between 10.3 and 13.0 feet NGVD, some with riprap toe protection, have been constructed seaward of the properties for all but approximately 530 feet of the shore. At the back of the Town Beach and along the road, upright granite slabs have been spaced to absorb some of the impact of storm waves but they do not prevent flooding. Castle Road, which runs parallel to the shoreline, provides access to the western part of the town. Further into the backshore are a number of mainly single family residences along Castle, Fox Hill and Ward Roads and Castle Way. A pharmacy is located at the corner of Castle and Ward Roads. The damage center is the low-lying and poorly drained area east of Castle Road.

Flooding occurs primarily due to the entry of storm waters through Town Beach and the properties without seawalls. Black Rock Beach has a slope of approximately 1 vertical and 10 horizontal. It is estimated that the crest elevation of 9.8 feet NGVD has a 5 percent chance of being overtopped each year. See Table B-9 in Appendix B.

Short Beach

The Study Area includes approximately 1200 feet of shoreline along Short Beach. Here the backshore is protected by a riprap revetment between the beach and Nahant Road, which provides the sole link between the town and the mainland. Further back is a commonly shared backshore with Black Rock Beach consisting of open space park land, a parking lot, the Lowlands Playground and undeveloped lowlands, including a sewer pumping station and an electrical sub-station. At the corner of Spring Road is a small commercial area. The riprap revetment has been modified to provide a walkway through it. The existing crest height is approximately 13.5 feet NGVD or capable of providing protection from a 50-year recurrence storm with wave runup (13.2 feet NGVD) and 0.4 feet below a 100-year recurrence storm with wave runup(13.9 feet NGVD). See Table B-10 in Appendix B. It is estimated that the crest would have between a 1 to 2 percent chance of being overtopped each year.

DRAINAGE

The overtopping of Black Rock and Short Beaches causes flooding and ponding in the residential and Lowlands Park backshore area. Floodwaters from this area discharge eventually into Nahant Harbor. A drainage ditch constructed parallel to Ward Road carries storm water under Flash Road and into the golf course and Bear Pond. From here it flows under Willow Road and the Pond Beach revetment and then through a double 24 inch pipe outfall equipped with tidal gates discharges into Nahant Harbor. When properly maintained, the gates prevent ocean backup into the golf course by closing during higher tidal stages. At lower tide stages the gates open to permit discharge of interior drainage water into Nahant Harbor. See Figures 1 and 2.

According to long term local residents, a drainage ditch carried storm water to an outfall pipe with a tidal gate under Black Rock Beach for discharge into Lynn Harbor. It was very likely located along the easement between Ward and Fox Hill Roads. See Figure 2. The ditch has since been filled in. Reportedly, the gates no longer function because of the accumulation of sedimentation at the outfall.

Chapter 3: PROBLEM IDENTIFICATION

STATEMENT OF PROBLEM

The Study Area is subject to flooding and ponding primarily as a result of coastal storms along Black Rock Beach. For more rare storm events flood waters enter the Study Area as a result of tidal surges and wave action along Short Beach. Details of expected tidal surges and wave actions under different storm conditions on the Study Area are presented in Appendix B.

Flooding occurs due to the overtopping of Black Rock Beach, with a minimum berm elevation of approximately 9.8 feet (NGVD) and direct rainfall. Short Beach has a minimum crest of 13.5 feet (NGVD). Although Black Rock Beach is partially sheltered due to its location on the inside of Lynn Harbor, the combination of significant tidal surge, wave action and low sand berm elevation at several stretches of the shoreline without seawalls and the Town Beach causes overtopping to occur beginning at approximately a 20-year storm event. The higher crested Short Beach, which is exposed to the open sea, would not be overtopped until storms achieve a severity of between the 50 and 100-year recurrence events. This conclusion is supported by local residents. They have observed that the more recent storms have induced flooding primarily through Black Rock Beach. The most devastating storms in recent history were the Blizzard of 1978 and the No Name or Halloween Storm of October 1991, which was the worst of four storms in the past three years. Based on the stillwater criterion, which is often used to characterize storms, the severity of storms, the 1978 Blizzard was a 100-year recurrence event and Halloween Storm a 17-year recurrence event.

The Study Area includes 53 structures (50 residential structures and land assessed at approximately $7.6 million, one pharmacy, a sewage pump station and an electrical sub-station) in the Lowlands Area of Nahant along Castle Road, Castle Way, Ward Road, Foxhill Road, and Harbor View Road. All but 10 of the structures have first floor elevations above the 100-year recurrence event. See Table 1 in Appendix A.

NO ACTION CONDITION

The no action condition is the most likely condition that is expected to occur in the Study Area in the absence of any public or private action to control flooding and to mitigate against the consequent damages to public and private property. The no action condition is a more severe future condition than the without project condition. In the no action condition, it is assumed that none of the actions expected by individuals or public agencies to protect or to remove property from the path of the flood would be undertaken.

WITHOUT PROJECT CONDITION

The without project condition is the most likely condition that is expected to occur in the Study Area during the 50-year period of evaluation in the absence of a federal project to improve conditions for public and private properties subject to flooding in the Black Rock and Short Beaches Study Area. In the absence of federal involvement, it is expected that no public or private entity would undertake measures to permanently and effectively reduce flooding in the Study Area. However, emergency measures would be initiated in the wake of storm conditions to protect life and to remove property from the path of flood waters.

Chapter 4: PLAN FORMULATION

PRELIMINARY SCREENING

All but approximately 530 feet of Black Rock Beach shoreline, including about 150 feet of Town Beach, is protected by a series of privately owned seawalls several of which are fronted with riprap for protection against less severe events. The back of the beach is set back from the alignment of the existing sea wall by approximately 50 feet. It is through those properties without seawalls and the Town Beach, that the tidal surges resulting from coastal storms discharge into the backshore lowlands and cause flooding of residential properties and open space. A number of alternatives have been considered for improving periodic flooding conditions in the Study Area as a result of the overtopping of Black Rock Beach during coastal storms. The overtopping of Short Beach during more rare storm events contributes to flooding in the Study Area.

A two step process is employed here in formulating improvements for controlling flooding in the Study Area. First, those alternatives that alleviate flooding through Black Rock Beach are considered. If a feasible solution at Black Rock Beach is identified, which has sufficient residual economic benefits, then improvements to Short Beach will be considered. Those alternatives for reducing flooding through Black Rock Beach would generally experience minimal flooding from direct rainfall and more significant flooding through Short Beach during more rare storm events. The alternatives may be structural, nonstructural or a combination of both.

<u>Structural Improvements</u>

Structural improvements reduce damages by altering the nature or extent of flooding. The so-called hard structural solutions, such as breakwaters, revetments, dikes and seawalls, generally are not favored by federal and state permitting agencies because of their possible deleterious effects on the subject and adjacent shorelines. In any case permits would have to be obtained in order to construct structural measures.

<u>Offshore Breakwater</u> - Since flooding along Black Rock Beach is caused by tidal surge and limited direct wave action, offshore structures to reduce wave energy, such as a breakwater, are not appropriate and have, therefore, not been considered.

<u>Revetment</u> - Revetments are placed on banks or bluffs to protect the shorelines behind them against light wave action and currents. Again, since the problem in the Study Area is flooding caused by tidal surge and not wave action, revetments are not considered further here.

Sandfill - Sandfill is a form of shore protection that can be placed on the shore to form a wider berm to prevent overtopping of the shore. Periodic nourishment is an integral part of a sandfill plan. Its purpose is to replace sand lost to offshore and littoral movement and to retain the geometric configuration and level of protection of the sandfill. A preliminary analysis indicated that sandfill protection to those reaches of the shoreline without seawalls to a 100-year level of protection warranted further investigation.

Dikes - Dikes are structures that isolate and protect areas from flooding. A dike was considered along Black Rock Beach but was eliminated from further consideration because of lack of economic feasibility.

Seawall - Like sandfills, seawalls can be used to protect shores from wave action and to separate the water from the land. They can range from robust structures used to protect shores from heavy wave action to more simple structures for sheltered shores, which is the case along Black Rock Beach, where large waves are rare. The analysis indicated that seawalls justified more detailed study.

Nonstructural Measures

Nonstructural measures decrease flood damages by reducing the vulnerability of public and private properties to flooding by changing the use of the flooded area. Among the several methods that are being considered for the study area are:

Raising of Existing Structures - Structures can be raised above the flood levels. The raising of structures is particularly applicable to single and two story frame houses such as are found in the Study Area. A number of residents in the Study Area have raised their houses.

Rearranging or Protecting Damageable Property - Damageable property can often be placed in a less vulnerable location. Furnaces and appliances, for example, can be protected by raising them to a higher base or higher floors or enclosing them in watertight utility rooms.

Relocation - Flood prone structures and/or contents can also be relocated outside the flood hazard area. Both structure and contents can be moved to a flood-free site or only the contents can be relocated and the structure demolished or converted to a new use at the existing site.

Flood Forecast, Warning and Evacuation - Flood forecasting, warning and evacuation is a strategy for reducing flood losses by designing a plan of action in response to the threat of flooding. The strategy includes:

- A system of early recognition and evaluation of flood potential, whether it be a community, automatic, National Weather Service or other broadcast or personal observation,

- Procedures for the issuance and dissemination of flood warnings should be made by responsible agencies and officials in a competent manner in order to motivate the people at risk to appropriate action.

- Evacuation of people and property including the mobilization of rescue, medical and fire squads and equipment, and the identification of priorities for evacuation,

- Maintenance of vital services, and

- Post-flood reoccupation of the flood prone area and recovery.

Approximately 10 homes would receive first floor flooding as a result of a 100-year recurrence storm event. A preliminary analysis indicates that federal participation in any of the above mentioned non-structural measures above would not be economically feasible. However, it is desirable for the town of Nahant to continue improving its procedures for providing residents of the flood prone area with as much advanced flood warning as possible in order to reduce the risk of injury to and loss of human life and to permit property to be temporarily raised above and/or removed from the flooded area. Given the loss of vehicles during former storms, their removal prior to storm events would seem to be in many cases an avoidable loss.

WITH PROJECT CONDITION

The with project condition is the most likely condition to occur in the Study Area if a federally assisted project is undertaken. There are as many with project conditions as there are alternative plans. The analysis focusses on the Black Rock Beach shoreline, which is primarily responsible for flooding in the Study Area except in rare events. Appendix C presents details of the design of the alternatives. Table 1 presents the first and annual costs of the alternatives.

<u>Sandfill Alternative</u>

<u>Design</u> - An approximately 600 foot length of sandfill would be constructed seaward of the properties without seawalls and along Town beach and feathered into adjacent seawalls and properties in the Black Rock Beach shoreline. The berm elevation has been rounded from 10.8 to 11.0 feet (stillwater level plus wave runup) to contain the 100 year storm event. The berm would be 10-feet wide with seaward slopes of 1 vertical and 10 horizontal. Subsequent to the determination of the structural

integrity of existing seawalls, the capping of the seawall may be necessary where their height is insufficient to provide the 100-year level of protection. Nourishment would also be necessary to periodically restore the geometric configuration of the sandfill and level of protection.

<u>Conditions With Sandfill</u> - The sandfill would minimize overtopping of Black Rock Beach to a negligible amount and would therefore reduce damages due to ocean flooding through Black Rock Beach except when the 100-year level of protection is exceeded. The sandfill options, however, have the disadvantage of sand washing onto the residential properties and into Castle Road during more severe storms. The sandfill's level of protection is sensitive to its timely restoration after a gradual or sudden loss of sand.

<u>Costs</u> - The estimated costs of the sandfill alternative are presented in Table 1. First construction cost, including contingencies and engineering and design and supervision and administration is estimated at approximately $296,000 and annual cost is $31,000.

<u>Seawall Alternative</u>

<u>Design</u> - Two reinforced concrete options are considered: a partial seawall and a complete seawall. See Appendix C for details.

The first seawall option would be approximately 580 feet long and constructed seaward of the residential properties without seawalls and along the backshore of Town Beach to a top elevation of 12.9 feet (NGVD) or protection against a 100-year recurrence event. It would be tied into the existing seawalls subsequent to a determination of its structural integrity. The existing seawall would then capped, as appropriate, to the elevation of the new construction. The design allows for a 45-foot segment to tie-in the portion of the seawall built seaward of the residences with the 150-foot long segment along the Town Beach backshore. The reinforced concrete wall and footing would have a thickness of 1.5 feet.

The second seawall option is a 1100-foot long seawall of the same characteristics as the first but constructed along the entire Black Rock Beach shoreline, including Town Beach, in the Study Area and not requiring the tie-in.

<u>Conditions With Seawall</u> - Like the sandfill options above, the seawalls would minimize the overtopping of Black Rock Beach and would reduce damages due to ocean flooding from Black Rock Beach up to the 100-year levels of protection.

Costs - The first costs of the partial and complete seawall options have been estimated respectively at $465,000 and $700,000, including contingencies, engineering, design, supervision, and administration.

Table 1
Black Rock and Short Beaches Shore Protection Study, Nahant, MA
COSTS OF ALTERNATIVES

Alternatives	Cost	E&D/ S&A* (15%)	First Cost	Amortization @ 7 3/4% (.07940)	Annual Maintenance/ Nourishment	Annual Cost
Partial Seawall	$405,000	$60,700	$465,700	$37,000	$3,500	$40,500
Complete Seawall	608,900	91,300	700,200	55,600	6,000	61,600
Partial Sandfill	257,400	38,600	296,000	23,500	7,500	31,000

* Engineering and Design/Supervision and Administration

ECONOMIC EVALUATION

Methodology

The economic analysis of the alternative shore protection plans is addressed in Appendix A. The potential benefits for each alternative as a result of the reduction of flood damages due to coastal storms are compared to the costs in 1994 prices. Benefits and costs are made comparable by conversion to average annual equivalents using an interest rate of 7 3/4 percent and a project economic life of 50 years. For each alternative annual benefits are divided by annual costs to determine the benefit-cost ratio. The ratio must be equal to or greater than one for federal participation in shore protection projects.

Results

Table 2 shows that none of the alternatives for controlling flooding in the Study Area are economically feasible since their benefit-cost ratios are less than one.

Table 2
Black Rock and Short Beaches Shore Protection Study, Nahant, MA
ECONOMIC ANALYSIS

Alternatives	Annual Benefits ($1,000)	Annual Costs ($1,000)	Benefit/ Cost Ratio
Partial Seawall	$20,100	$40,500	0.5
Complete Seawall	$20,100	$61,600	0.3
Partial Sandfill	$20,100	$31,000	0.6

Chapter 5: FINDINGS AND DETERMINATION

This Section 103 Reconnaissance Study concludes that the considered alternative plans to control flooding in the Black Rock and Short Beaches Study Area are not economically justified.

A chronic flooding problem generated by coastal storms exists in the Study Area. Black Rock Beach is overtopped under more frequent storm events and Short Beach under more rare storm events causing flooding of the residential and open space backshore area.

A two step process was employed in formulating improvements for controlling flooding. First, those alternatives that would alleviate flooding through Black Rock Beach were considered. Improvements across Short Beach would then be identified and evaluated if a feasible solution along Black Rock were identified with sufficient residual economic benefits. Three alternative plans were formulated and evaluated in detail for controlling flooding generated primarily by tidal surge across Black Rock Beach. Since none of the alternative plans would produce benefits in excess of costs, their benefit-cost ratios were less than the required ratio of 1.0. No improvement plans were, therefore, found to be economically feasible in controlling flooding conditions emanating from Black Rock Beach. These same conclusions consequently pertain to Short Beach.

I have therefore determined that there is no opportunity for Corps of Engineers' assistance in the Study Area and work is terminated with this report.

23 March 1995
Date

Earle C. Richardson
Colonel, Corps of Engineers
Division Engineer

Chapter 6: ACKNOWLEDGEMENTS

The New England Division, Army Corps of Engineers prepared this report under the direction of Colonel Earle C. Richardson, Division Engineer, and under the supervision of Mr. John T. Smith, Chief, Coastal Development Branch, Mr. Paul E. Pronovost, Chief, Plan Formulation Division and Mr. Joseph L. Ignazio, Director of Planning.

This report was prepared by Mr. Charles L. Joyce, Project Manager, with assistance from the following members of the Study Team:

Ms. Karen Umbrell - Economic Analysis
Mr. Marc Paiva - Historical\ Archaeological Resources
Mr. Jay Mackay - Environmental Resources
Mr. Robert Meader - Coastal Engineering
Ms. Carmen Suarez - Hydrologic Engineering
Ms. Heather Sullivan - Hydraulics
Mr. Paul Young - Geotechnical Engineering
Ms. Sheila Bergeron - Geotechnical Engineering
Mr. Christopher Lindsay - Cost Engineering
Ms. Maureen McCabe - Real Estate
Mr. Edward Fallon - Real Estate

The report was prepared for publication by Ms. Kristina Reitz.

Appreciation is extended to the Town of Nahant Board of Selectmen and the Town Administrator and his staff for their assistance and cooperation during the course of the study. A special thanks to Mr. Thomas Gallery, the town's Emergency Management Director, who provided invaluable knowledge and assistance to Study Team Members.

II APPENDICES

APPENDIX A
ECONOMIC ANALYSIS

Appendix A
Economic Analysis

Lowlands Area
Nahant, Massachusetts
Section 103 Reconnaissance Study

Introduction

The purpose of this analysis is to identify and evaluate the economic feasibility of providing flood damage protection to the Lowlands area of Nahant, Massachusetts. The town of Nahant is located on the eastern coast of Massachusetts, approximately ten miles north of Boston. Nahant is actually made up of two islands, the smaller of which is known as Little Nahant, which is located between the mainland and the larger Nahant island. The islands are connected to mainland Massachusetts by a long causeway. The Lowlands area is located on the western edge of the larger island. The area has suffered severe coastal flooding in the past, with the most severe flooding having occurred in the storm of 1978. The area also experienced flooding in the coastal storms of October 1991 and December 1992.

This economic analysis includes a description of the study area, estimates of the recurring and expected annual flood damages for the study area, calculation of the annual benefits derived from preventing those damages, and a determination of the economic justification of the proposed improvement plan by calculating the benefit to cost ratio of the plan.

Methodology

A proposed project is considered economically justified if it has a benefit to cost ratio equal to or greater than 1.0, that is, if the benefits of the project equal or outweigh the costs of the project. In a general sense, the economic benefits of a project are determined by comparing the without project condition to the with project condition, and evaluating the difference between the two conditions.

In accordance with Corps of Engineers guidelines, the benefit to cost ratio for the improvement plan examined is calculated by comparing the benefits and costs of the plan in average annual equivalent terms. Costs and benefits are converted to average annual equivalents based on the fiscal year 1995 federal interest rate for water resources projects of 7 3/4 percent, and based on a 50 year period of analysis. All costs and benefits contained in this analysis are stated at the 1994 price level, and the analysis is performed at the reconnaissance level of detail.

Description of the Study Area

The Lowlands area of Nahant is located on the eastern corner of the larger, main island of Nahant. The study area includes structures located on Castle Road, Castle Way, Ward Road, Fox Hill Road, and Harbor View Road. The study area contains a total of 53 structures which are located within the 100 year flood plain. Of those 53 structures, 50 are residential structures, 1 is a commercial structure - a pharmacy, and 2 are public structures - a pump station and an electrical sub-station. Of the 50 residential structures, 48 are single family homes and 2 are multi-family homes. The houses are primarily year-round residences, of one and two stories, and are generally in good to excellent condition. In all, the 50 houses in the study area have a total assessed value, including land and structures, of approximately $7.6 million.

Economic Setting

Located close to the city of Boston, Nahant is part of the Greater Boston Metropolitan Area. In general, the town is primarily a residential suburb, with only a small amount of retail development, and with the vast majority of its residents employed in Boston and the surrounding cities and towns. According to the 1990 US Census, in 1990 Nahant had a total population of 3,828, and contained 1,687 housing units.

Existing Conditions

The Lowlands area suffered the most severe damages in the severe winter storm of 1978. In that storm, many houses suffered severe flooding damages, and roads were impassable. More recently, the area was flooded in the storm which occurred on October 31, 1991. After that storm, the area had to be pumped with several large pumps for five days before water levels were reduced.

Without Project Condition

It is projected for the without project condition that no measures providing permanent, effective protection to the study area from flood damages will be constructed in the study area by town, state, or federal interests. As a result, in the without project condition, it was assumed that flood damages similar to those that have been experienced in the past will continue to be experienced in the study area in the future.

With Project Condition

Three alternative measures to reduce flooding in the study area were examined in this analysis - partial seawall, complete seawall, and partial sandfill. Details regarding these alternatives are contained in the main report. All three of these alternatives would significantly reduce the flooding in the study area that is caused by elevated ocean levels and wave overtopping.

Structure Inventory

The first step in the economic analysis was to categorize the structures located within the 100 year flood plain by elevation and type of structure. The 100 year flood plain in the Lowlands area was identified based on the Flood Insurance Administration floodplain maps produced by the Federal Emergency Management Agency. A survey of the structures in the 100 year flood plain was then conducted to determine the first floor elevations of the structures. The 53 structures in the flood plain have first floor elevations distributed as shown in Table 1, below.

Table 1
Distribution of First Floor Elevations
Lowlands Area, Nahant, MA

Elevation (feet)								
6	7	8	9	10	11	12	13+	Total
1	0	3	2	4	7	17	19	53

The structures in the study area were categorized by type and size, such as Small 1 Family 1 Story, Small 1 Family 2 Story, Large 1 Family 2 Story, etc.. In order to estimate the flood damage that would occur to the structures at various flooding levels, typical stage-damage functions were used. Different typical damage functions were used for each category of structure. The damages in the typical stage-damage functions are estimated in one foot increments, from the basement up to six feet above the first floor. The damage functions include damage estimates for structural damage, damage to contents, damage to utilities, damage to outside grounds, and estimated non-physical losses such as costs for temporary relocation during a flood. For the commercial structure and the two public structures in the study area, stage-damage functions were estimated using depth-damage relationships developed in previous Corps of Engineers studies for similar structures.

Recurring Losses

The stage-damage functions assigned to the structures in the study area were then aggregated, based on first floor elevations, to determine the total stage-damage function for each hydrologic zone in the study area. Based on the hydrologic analysis that was performed for this study, the study area was divided into two zones, a Castle Road zone, and the remainder of the study area. For each of the two zones, stage frequency curves for both the without-project condition and the with-project condition were developed. The with-project condition curves apply to all three of the improvement alternatives being examined.

The total stage-damage curve for each zone was then combined with the stage-frequency curve for each zone, in order to determine the damage-frequency relationships under both the without and with project conditions.

The damages that are expected to occur under existing conditions at flood events of different frequencies are termed the recurring losses for the study area. The recurring losses under the existing conditions for a range of events are shown in Table 2, below. The figures shown include both hydrologic zones and thus represent the entire study area.

Table 2
Recurring Losses
Exiting Conditions
Lowlands Area, Nahant, MA

Probability (%)	Frequency (year)	Estimated Damages ($)
50	2	$ 13,500
20	5	$ 25,100
10	10	$ 38,700
5	20	$ 84,500
2	50	$332,600
1	100	$598,700

Expected Annual Damages

Expected annual damages are calculated by multiplying the predicted damages for each flood event by the annual percentage chance that each event will occur. The resulting expected damage at each event, given each event's probability of occurrence, are then added together to yield the total expected annual damages for the study area. The expected annual damage figure represents the average annual damage that could be expected to occur based on the weighted probabilities of the complete range of storm events. The without project condition stage frequency curves for each zone were combined with the stage-damage curves for each zone, and the total expected annual damages for the study area under the without project condition were calculated to equal $34,200.

For analyzing the with project condition, which would apply to any of the three improvement alternatives being analyzed, the with-project stage-frequency curves were combined with the stage-damage curves for each zone, and the expected annual damages were calculated to equal $14,100.

Calculation of Benefits

In Corps of Engineers flood damage reduction studies, the effectiveness of a flood damage reduction plan is measured by the extent to which it reduces expected annual damages. In order to calculate the annual benefits of any of the three proposed improvement alternatives, the expected annual damages under the without project condition are compared to the expected annual damages under the with project condition. The difference between the two expected annual damage figures equals the annual benefits for the alternatives being examined. With expected annual damages under existing conditions of $34,200, and expected annual damages under the with project condition of $14,100, the total annual benefits equal $20,100 ($34,200 - $14,100 = $20,100).

Economic Justification

In order for a federal project to be considered economically justified, it must have a benefit to cost ratio equal to 1.0 or greater. Additional information regarding the costs of the alternatives is contained in the main report and the Engineering Appendix. The annual benefits, annual costs, benefit to cost ratio, and net annual benefits for the alternatives being examined are shown in Table 3, below. As can be seen in Table 3, it was determined in this analysis that none of the alternatives show economic justification.

Table 3
Economic Justification

Plan	Annual Benefits	Annual Costs	Benefit to Cost Ratio	Net Annual Benefits
Partial Seawall	$20,100	$40,500	0.5	none
Complete Seawall	$20,100	$61,600	0.3	none
Partial Sandfill	$20,100	$31,000	0.6	none

APPENDIX B
HYDROLOGY AND HYDRAULICS

APPENDIX B

HYDROLOGY AND HYDRAULICS
COASTAL FLOOD REDUCTION STUDY
BLACK ROCK AND SHORT BEACHES
NAHANT, MASSACHUSETTS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

APPENDIX B

HYDROLOGY AND HYDRAULICS
COASTAL FLOOD REDUCTION STUDY
BLACK ROCK AND SHORT BEACHES
NAHANT, MASSACHUSETTS

1. PURPOSE

This reconnaissance report presents the results of studies concerning coastal flooding conditions of the Lowland Park area in the town of Nahant, Massachusetts. The study was performed under authority contained in Section 103 of the 1948 Flood Control Act. Flooding in this area is caused mainly by ocean wave overtopping and poor interior drainage conditions. Rainfall contributes in a much lesser extent to the flooding effects on streets adjacent to the "Lowlands Park area". Included in the report are a general description of the area, climatology, rainfall analysis and sections concerning tidal and interior hydrologic analysis.

2. DESCRIPTION

Located on a peninsula, in the northeastern end of Boston Harbor, the town of Nahant connects to the mainland by a manmade causeway, built on a geologic formation. This causeway, the Nahant Beach Parkway, is owned and maintained by Massachusetts District Commission (MDC). Neighboring cities to Nahant are Swampscott and Lynn to the north, Revere and East Saugus to the west, and Winthrop and Boston to the south. Nahant is moderately urbanized, mostly by single residential buildings with minor local commercial buildings. The study area is shown on plate B-1.

The study area is located in the site locally known as the "Lowlands Park Area," in the northwestern corner of Nahant (see plate B-2). This area is comprised mostly of single family homes with minor commercial development. Two beaches--Black Rock along Lynn Harbor, and Short along Nahant Harbor surround the project site. Overtopping along a third one--Town Beach, on the Lynn Harbor side, also contributes to flooding, however, this beach is sometimes considered part of Black Rock. For this reason and due to its size, approximately 150-foot long, Town Beach will be considered part of Black Rock for the present report. The Lowlands area does not experience flooding during normal high tides.

Black Rock Beach is approximately 5,200 feet long. A rock revetment in its northern section (2,700 feet), ranging in height between 13.0 and 14.0 feet NGVD, reduces wave overtopping over the Nahant Beach Parkway. With a few exceptions the houses on Black Rock Beach are fronted by privately-owned seawalls, ranging in top elevations between 10.3 and 13.0 feet NGVD. Seven houses lack any type of protection. Short Beach, located along Nahant Harbor, is roughly 2,800 feet long, with revetment ranging between 13.0 to 14.5 feet NGVD. This beach is open to Nahant Beach Parkway and a U.S. Coast Guard station is located there. Wave overtopping in the lower 800 feet of this beach occurs only for rare events.

During major storms, Black and Short Beaches are overtopped by wind-generated waves that flood interior areas. Wave overtopping and rainfall runoff run through the streets and drain into the Lowlands Park, which has the lowest elevations in the study area serving as storage basin for floodwaters. Formerly, a 36-inch drain pipe located along an easement section near the intersection of Castle and Ward Roads drained water concentrated in the Lowlands Park into Lynn Harbor. However, this drain is no longer operable. The principal drainage system for the area is a trench and 36 inch-pipe combination located in the Lowlands Park, which discharges into Bear Pond. This pond, in the southern end of Nahant, discharges into Pond Beach by a double 24-inch culvert equipped with tidal gates.

3. CLIMATOLOGY

a. General. The town of Nahant lies in the path of prevailing "westerlies", and has a semi-humid variable climate typical of New England. Due to direct coastal exposure, it frequently experiences periods of intense precipitation, produced by local thunderstorms and large weather systems of tropical or extratropical origin such as "northeasters," producing high tides and waves common to the New England area. The area has an annual mean temperature of 51.5 degrees Fahrenheit; mean monthly temperatures vary between 73.5 in July to 29.6 in January. Monthly climatological information developed at Boston is considered applicable for Nahant and shown in table B-1.

b. Precipitation. Mean annual precipitation at Boston Logan Airport has been recorded to be 41.68 inches. Short duration intense rainfall often accompanies fast moving frontal systems, thunderstorms, and coastal storms. Precipitation is distributed quite uniformly throughout the year, averaging about 3.5 inches per month. Peak storm rainfall frequency duration data as reported in U.S. Weather Bureau Technical Paper 40 is summarized in table B-2. Also, Boston's average annual snowfall of 40.55 inches occurs primarily from December through March.

TABLE B-1

MONTHLY CLIMATOLOGIC DATA
BOSTON WSO AP

Period of Record 1962-1993)

Month	Precipitation	Temperature			Snowfall	Prevailing Wind	
	Mean (in.)	Mean	Max (oF)	Min	Mean (in.)	Mean (mph)	Direction
January	3.62	29.60	63	-12	12.20	13.9	NW
February	3.40	30.7	70	-4	12.30	13.8	WNW
March	3.88	38.40	81	6	7.60	13.8	NW
April	3.71	48.7	94	16	0.97	13.2	WNW
May	3.20	58.5	95	34	0.02	12.2	SW
June	3.31	68.00	100	45	0.00	11.5	SW
July	3.03	73.50	102	50	0.00	11.0	SW
August	3.48	71.90	102	47	0.00	10.8	SW
September	3.13	64.60	100	38	0.00	11.3	SW
October	3.13	54.80	90	28	0.00	12.0	SW
November	4.02	45.2	78	15	1.24	12.9	SW
December	3.83	33.7	73	-7	7.27	13.6	WNW
TOTAL	41.68	51.5	102	-12	40.55	12.5	SW

TABLE B-2

RAINFALL FREQUENCY DURATION
USWB TECHNICAL PAPER 40
BOSTON MASSACHUSETTS
(Inches)

Annual (%)	Frequency (year)	Duration in Hours 1	2	6	12	24
50	2	1.1	1.5	2.1	2.6	3.1
20	5	1.5	2.0	2.8	3.4	4.0
10	10	1.8	2.3	3.3	3.9	4.6
2	50	2.4	3.1	4.3	5.1	6.0
1	100	2.6	3.3	4.7	5.8	6.8

4. TIDAL HYDROLOGY

a. Astronomical Tides. In the study area (figure B-1), tides are semidiurnal, with two high and two low waters occurring during each lunar day (approximately 24 hours, 50 minutes). The resulting tide range is constantly varying in response to relative positions of the earth, moon, and sun, with the moon having the primary tide producing effect. Maximum tide ranges occur when orbital cycles of these bodies are in phase. A complete sequence of tide ranges is repeated over an approximate interval of 19 years, known as a tidal epoch.

(1) Boston. At the National Ocean Survey (NOS) tide gage in Boston, Massachusetts (the one nearest to the study area), the mean range and mean spring range of tides are 9.5 and 11.0 feet, respectively. However, maximum and minimum predicted astronomic tide ranges at Boston have been estimated at about 14.7 and 5.0 feet, respectively, using the Coastal Engineering Research Center (CERC) report, entitled "Tides and Tidal Datums in the United States," SR No. 7, 1981. The frequency of astronomic tidal fall (the difference between consecutive high and low tides), as determined by CERC, is presented in figure B-2. The variability of astronomical tide ranges is a very significant factor in tidal flooding potentials throughout the area under study, and is further explained in section 4d.

Because of continual variation in water levels due to tides, several reference planes, called tidal datums, have been defined to serve as reference points for measuring elevations of both land and water. Tidal datum information for Boston is presented on figure B-3 and table B-3. The data were compiled from currently available NOS tidal

STUDY AREA VICINITY

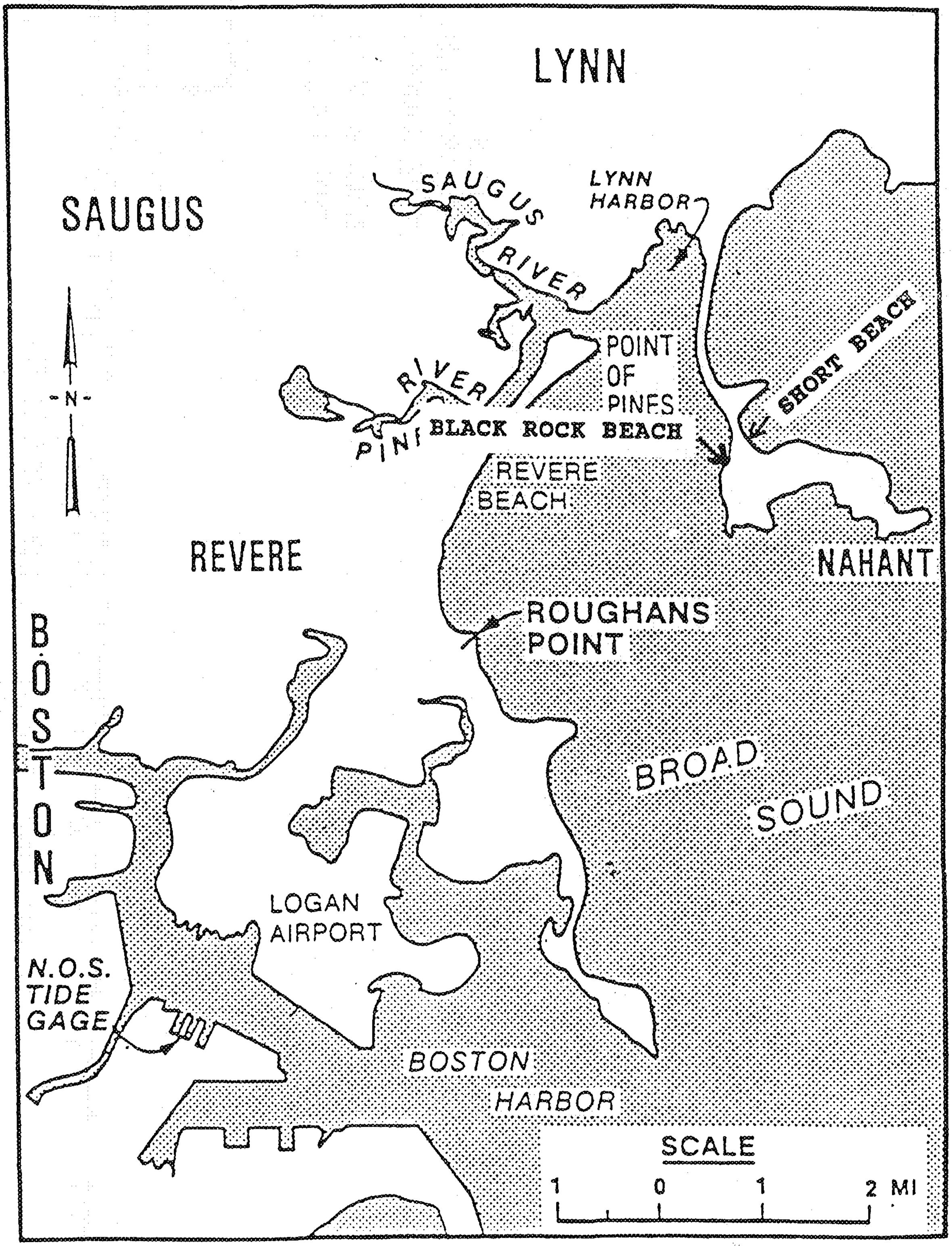


FIGURE B-1

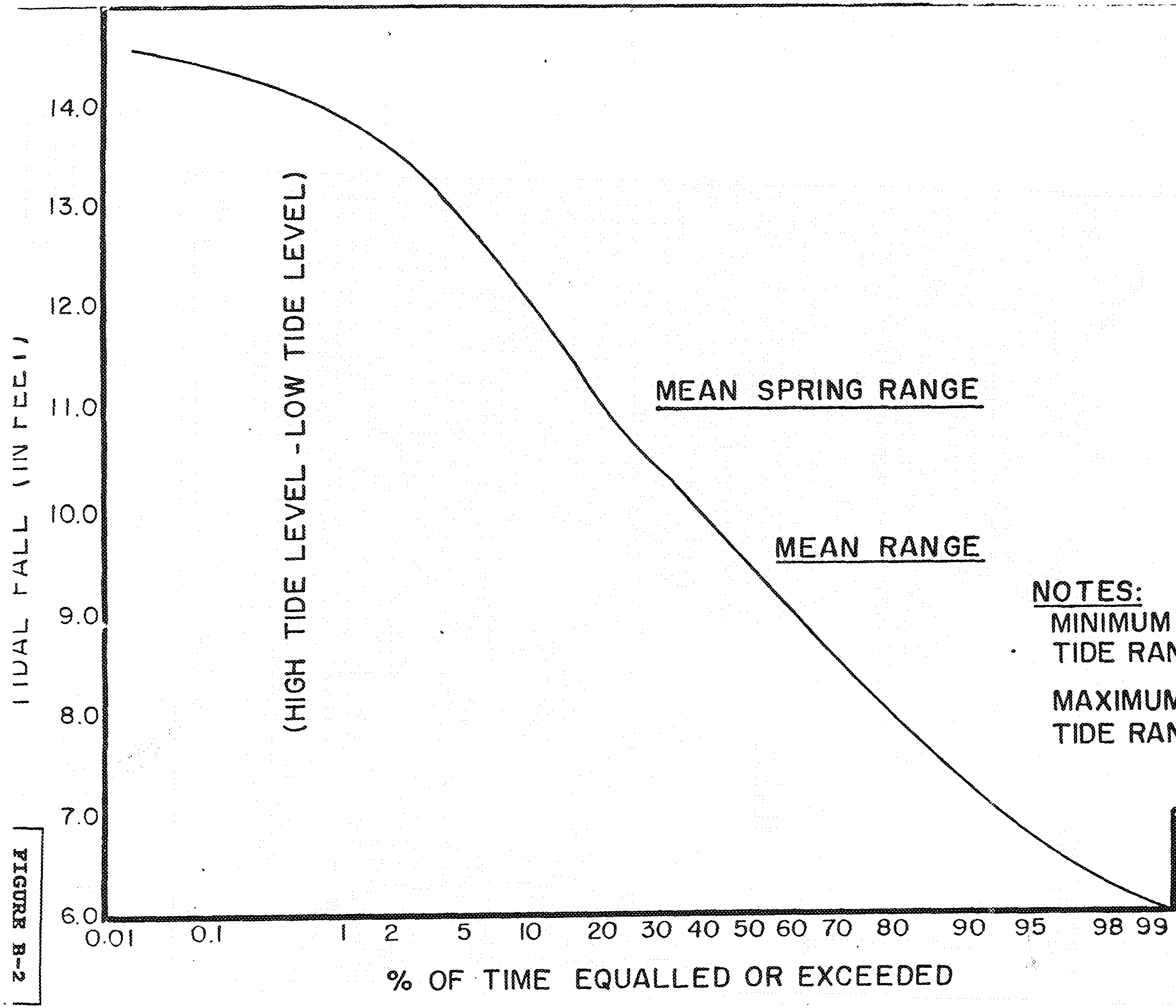
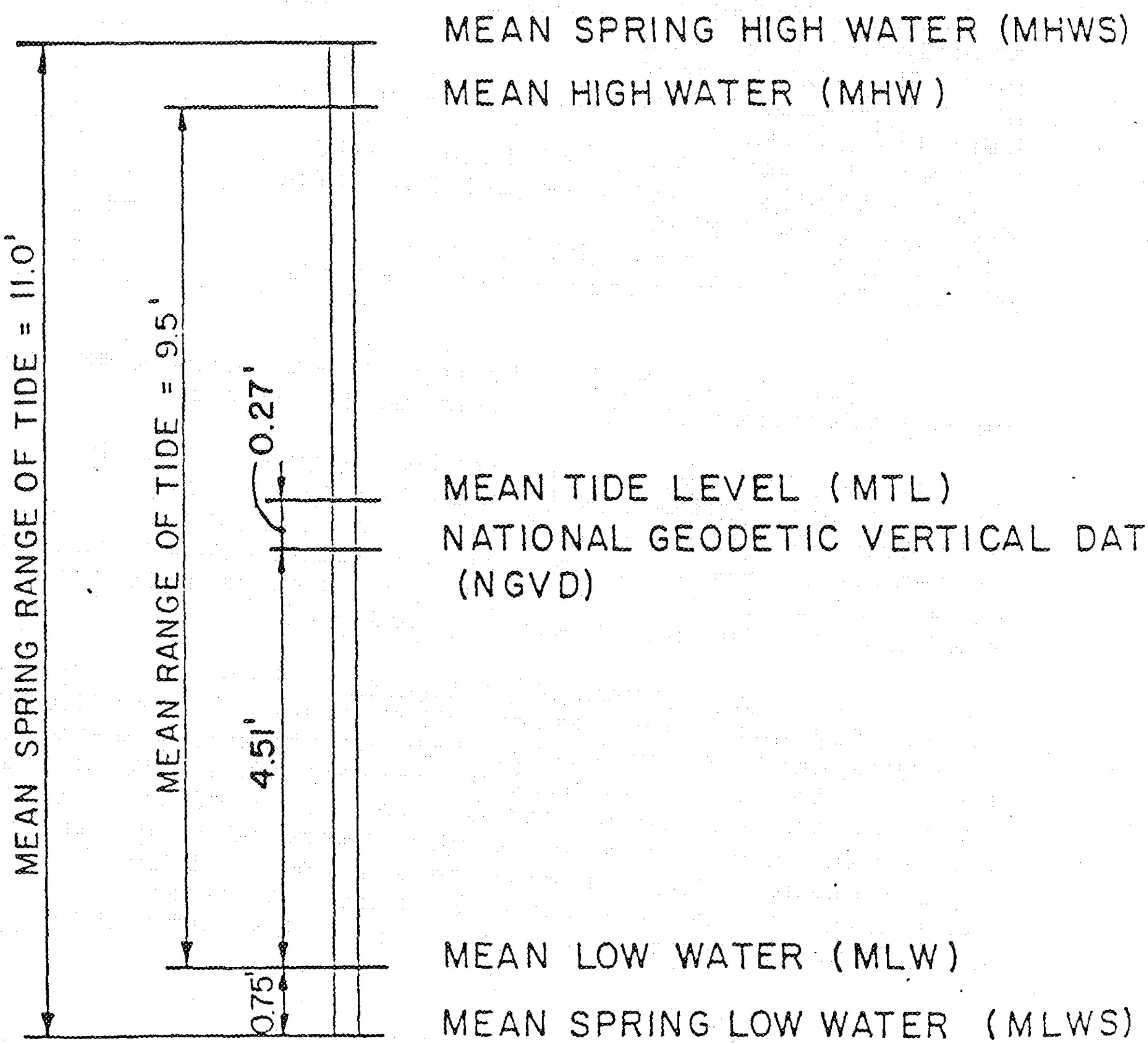


FIGURE B-2

FREQUENCY OF TIDAL FALL
BOSTON, MA.
HYD.& W.Q.SECT.
JUNE 1988

TIDAL DATUM PLANES
BOSTON, MASSACHUSETTS
NATIONAL OCEAN SURVEY TIDE GAGE

(BASED UPON 1960-78 NOS TIDAL EPOCH)



NEW ENGLAND DIVISION
U.S. ARMY, CORPS OF ENGINEERS
WALTHAM, MASS.

FIGURE B-3

benchmark information for Boston, along with the previously mentioned CERC report. The epoch for which the National Ocean Survey has published tidal datum information for Boston is 1960-78. A phenomenon that has been observed through tidal gaging and benchmark measurements is that sea level is apparently rising, with respect to land along most of the U.S. coast. At the Boston National Ocean Survey tide gage, the rise has been slightly less than 0.1 foot per decade. Sea level determination is generally revised at 25-year intervals to account for the changing sea level phenomenon.

TABLE B-3

BOSTON TIDAL DATUM PLANES
NATIONAL OCEAN SURVEY TIDE GAGE
(Based Upon 1960-78 NOS Tidal Epoch)

	Tide Level (ft, NGVD)
Maximum Predicted Astronomical High Water	7.5
Mean Spring High Water	5.8
Mean High Water (MHW)	5.0
Minimum Predicted Astronomical High Water	2.7
Mean Tide Level (MTL)	0.3
National Geodetic Vertical Datum (NGVD)	0.0
Maximum Predicted Astronomical Low Water	-2.4
Mean Low Water (MLW)	-4.5
Mean Spring Low Water (MLWS)	-5.2
Minimum Predicted Astronomical Low Water	-7.1

(2) Study Area. In the Nahant study area, tides are nearly identical to those observed at Boston.

b. Storm Types. Two distinct types of storms, distinguished primarily by their place of origin as being extratropical and tropical cyclones, influence coastal processes in New England. These storms can produce above normal water levels and waves, and must be recognized in studying New England coastal problems.

(1) Extratropical Cyclones. These are the most frequently occurring variety of cyclones in New England. Low pressure centers frequently form or intensify along the boundary between a cold, dry continental air mass and a warm, moist marine air mass just off the coast of Georgia or the Carolinas, and then move northeastward more or less parallel to the coast. These storms derive energy from the temperature contrast between cold and warm air masses. The organized circulation pattern associated with this type of

storm may extend for 1,000 to 1,500 miles from the storm center. The wind field in an extratropical cyclone is generally asymmetric with the highest winds in the north-eastern quadrant. When the storm center passes parallel and to the southeast of the New England coastline, and highest onshore windspeeds are from the northeast, these storms are called "northeasters" or "nor'easters" by New Englanders. As the storm approaches and passes, local wind directions may vary from southeast to slightly northwest. Coastlines exposed to these winds can experience high waves and extreme storm surges. Such storms are the principal tidal flood-producing events throughout the study area. Other storms, taking a more inland track, can have high winds from the southeast and are referred to as "southeasters." In the area under study, these storms do not generally produce as much storm surge and wave action as "northeasters" due to more limited fetch. November through April is the prime season for severe extratropical storms in New England.

(2) Tropical Cyclones. These storms form in a warm moist air mass over the Caribbean and waters adjacent to the West Coast of Africa. The air mass is nearly uniform in all directions from the storm center. Energy for the storm is provided by the latent heat of condensation. When the maximum windspeed in a tropical cyclone exceeds 75 mph, it is labeled a hurricane. Wind velocity at any position can be estimated, based upon the distance from the storm center and forward speed of the storm. The organized wind field may not extend more than 300 to 500 miles from the storm center. Recent hurricanes affecting New England generally have crossed Long Island Sound and proceeded landward in a generally northerly direction. However, hurricane tracks can be erratic. The storms lose much of their strength after landfall. For this reason, the southern coast of New England experiences the greatest surge and wave action from strong southerly to easterly flowing hurricane winds. However, on very rare occasions, reaches of coastline in eastern and northern New England may experience some storm surge and wave action from a weakened storm. Hurricanes have not been a principal cause of tidal flooding in the greater Boston area. The hurricane and tropical storm season in New England generally extends from August through October.

c. Winds. An estimate of windspeed is one of the essential ingredients in predicting wave heights. The most accurate estimate of winds at sea, which generate waves and propel them landward, is obtained by utilizing isobars of barometric pressure recorded during a given storm. However, actual recorded windspeed and direction data, observed at a land-based coastal meteorological station, can serve as a useful guide when more locally generated waves and currents are of interest. The disadvantage of using land-based wind records is that they may not be totally indicative of wind velocities at the sea-air interface where waves are generated. However, often they are the only available source of information, and adjustments must be made to develop over-water estimates from land-based records. Also, when estimating wave overtopping of coastal structures, it is necessary to utilize local wind conditions. These local winds help determine how much runup from breaking waves is blown over the structure.

Wind information presented in the following three paragraphs was developed for Revere, and is presented in the 1989 Feasibility Report for Saugus River and Tributaries, Lynn, Malden, Revere, and Saugus, Massachusetts. This information is applicable to Short Beach, which has fetch directions similar to Revere, but does not apply to Black Rock Beach.

(1) Percent Occurrence of Wind Direction and Speed. The National Weather Service (NWS) has recorded 31 years of hourly, 1-minute average windspeed and direction data at Logan International Airport in Boston, Massachusetts, from 1945 through 1979. Logan Airport, adjacent to the study area, is the closest location to the project where relatively complete, systematically recorded wind data are available. Windspeed data for this period alone were adjusted to a standard 33-foot observation height, and 1-minute average windspeeds were converted to 1-hour average windspeeds. Since Logan International Airport is almost directly adjacent to the ocean, no land-to-sea conversion was applied. However, a wind stability correction was made for all fetches of interest. All adjustments were made in accordance with ETL 1110-2-305 on the subject of determining wave characteristics on sheltered waters. Utilizing these one-hour average wind data, the percent occurrence of wind direction and windspeed range were computed. This analysis, with results shown in table B-4 and figure B-4, indicated that the principal onshore wind direction for windspeeds ≤5 mph is from the SE, and for windspeeds >5 and ≤15 mph, from ESE. Winds at >15 and ≤20 mph generally arrive from the east, and >20 mph from the NE. The maximum average windspeed (11.8 mph) is from the NE, and the greatest maximum speed, 68.7 mph, from the SE. Overall average speed is 10.5 mph. Table B-4 also shows resultant wind direction for various windspeed ranges. The resultant wind direction is a vector quantity computed using the product of windspeed and direction, and is an indicator of net air movement past a given location. Overall, the resultant wind direction is from the east. However, winds >20 mph have a more ENE resultant. The greatest percentage of windspeeds is shown to be >10 and ≤15 mph.

TABLE B-4

ADJUSTED HOURLY WIND OBSERVATIONS BETWEEN NE AND SE
AT BOSTON, MASSACHUSETTS
(One-Hour Average Values)

PERCENT OF ONSHORE WINDSPEED AND DIRECTION OBSERVATIONS (X 10)

Direction	Windspeed Range (Miles Per Hour) 0-5	5-10	10-15	15-20	20-25	25-30	30-35	Over 35	All Inclusive	Avg Speed (mph)	Max Speed (mph)
NE	19	46	55	31	16	8	3	2	179	11.8	54.3
ENE	20	52	59	31	13	7	2	2	185	11.3	49.2
E	23	69	91	33	10	5	2	1	234	10.7	55.3
ESE	22	73	92	30	7	2	1	0	227	10.0	49.2
SE	24	72	63	13	2	1	0	0	174	8.7	68.7
NE-SE	108	313	360	136	48	22	7	5	1,000	10.5	68.7
Resultant Direction:	E	E	E	E	ENE	ENE	ENE	ENE	E		

NOTES: 1. Windspeed ranges include values greater than the lower limit and less than or equa to the higher limit.

2. Onshore winds occur 21 percent of the time; therefore, average annual number of occurrence (A) = percent occurrence times 18.654 (for example: a windspeed range of 0-5 mph from the ENE, A = 2.0 (18.654) = 37).

BOSTON, MA, ONE-HOUR AVERAGE ONSHORE WINDS

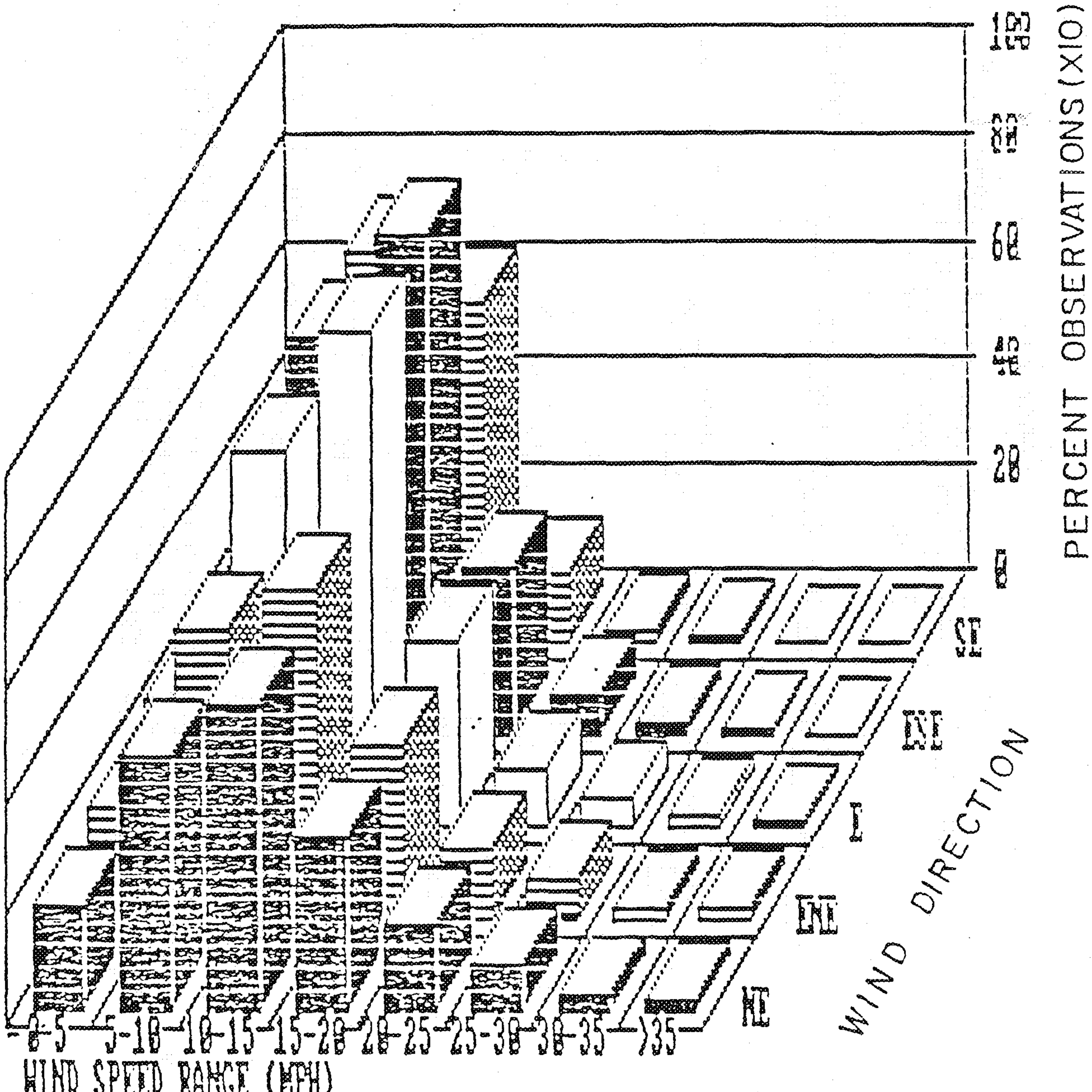


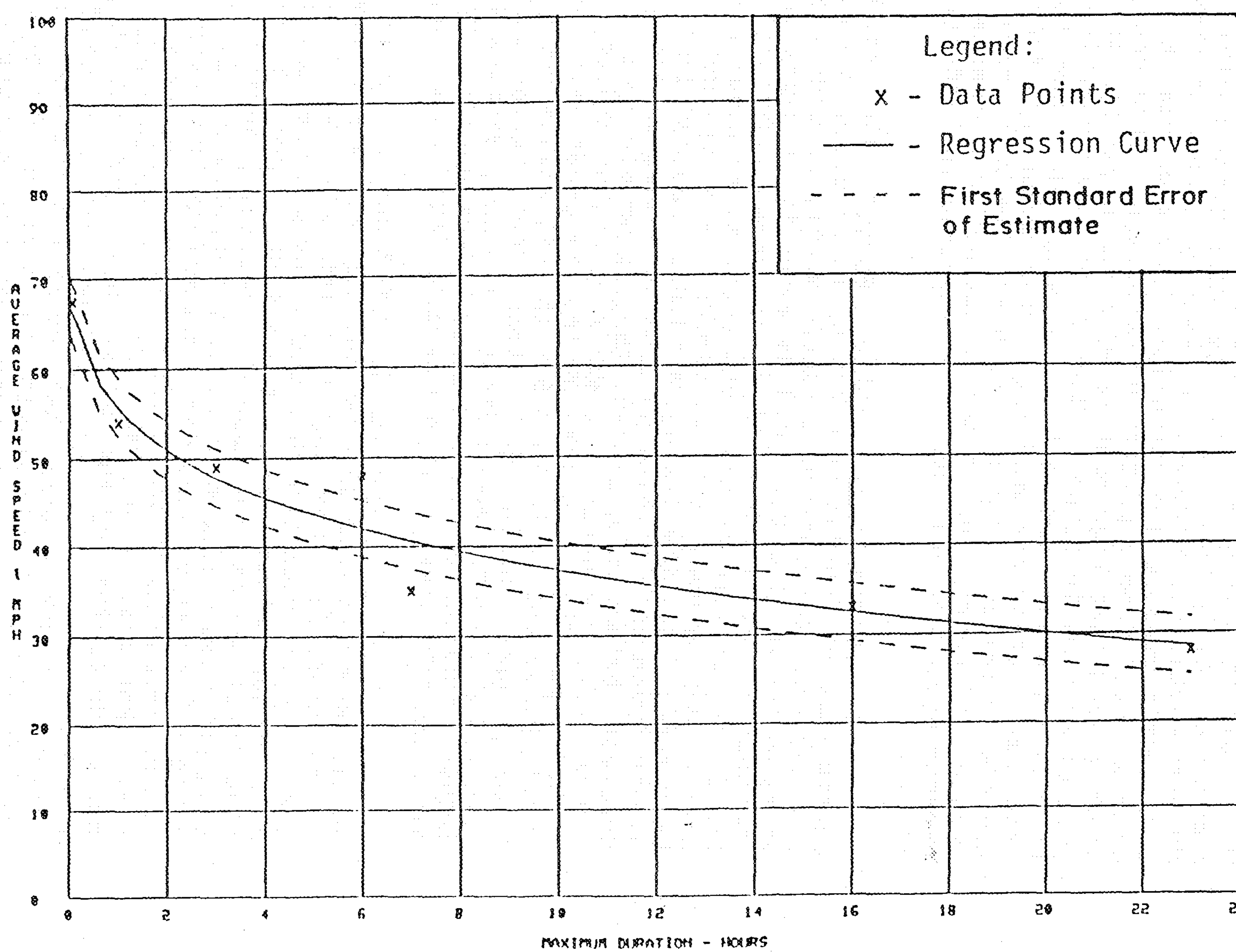
FIGURE B-4

(2) Windspeed Persistence. Additionally, actual windspeed persistence was determined on a directional basis. The resulting maximum windspeed persistence data, shown on figures B-5(a) through B-5(e), for directions northeast through southeast, indicate the maximum number of consecutive hourly windspeed observations that occurred at a given average speed from a particular direction. This analysis demonstrated that high onshore wind can occur for extended periods of time in Boston. High speed, long duration winds are usually associated with northeasters and, therefore, come from the northeastern quadrant. High intensity, short duration winds are from the southeast due to hurricane events. Of course, winds far out at sea can possess much greater speed and duration than reflected in land-based records.

(3) Winds During Historic Storms. When studying overtopping of coastal structures, it is useful to examine wind conditions during past flood events, in order to obtain an appreciation for the possible severity of experienced wave overtopping conditions. Table B-5 presents National Weather Service (NWS) wind observations recorded at Logan Airport, in Boston, during notable tidal floods. From the data, it can be observed that the strongest winds, recorded during flood events, generally originated from northeast to east. The greatest fastest mile listed, (approximately equal to 1-minute average speed), 61 mph from the northeast, was recorded on 6 February 1978 during the great "Blizzard of '78." By comparing table B-5 with B-8, stillwater tide levels recorded during these storm events ranged between 10.4 and 8.2 feet, respectively. However, extremely severe onshore winds have occurred during storm events, producing significantly lower observed maximum stillwater tide levels.

Since the astronomical tide range at the project is so variable, as explained in section 4a, many severe coastal storms occur during periods of relatively low astronomic tides. Thus, even though a storm may produce exceptionally high onshore winds, waves, and a tidal surge, the resulting tide level may be less than that occurring during a time of high astronomic tide and little meteorological influence. Table B-6 presents wind data recorded at Logan Airport during storms producing annual maximum surge values of 3 feet or more. For comparison, table B-7 lists maximum annual storm surges determined by the NWS in their "Tide Climatology For Boston, MA," November 1982, and associated observed tide levels. It can be seen that recurrence intervals of the maximum observed tide levels, recorded on days of maximum annual storm surge, were generally less than one year, with only a few storms producing significant tidal flood levels. (There may be some slight discrepancies in levels reported by the NWS in 1982, and more recent data from NOS due to

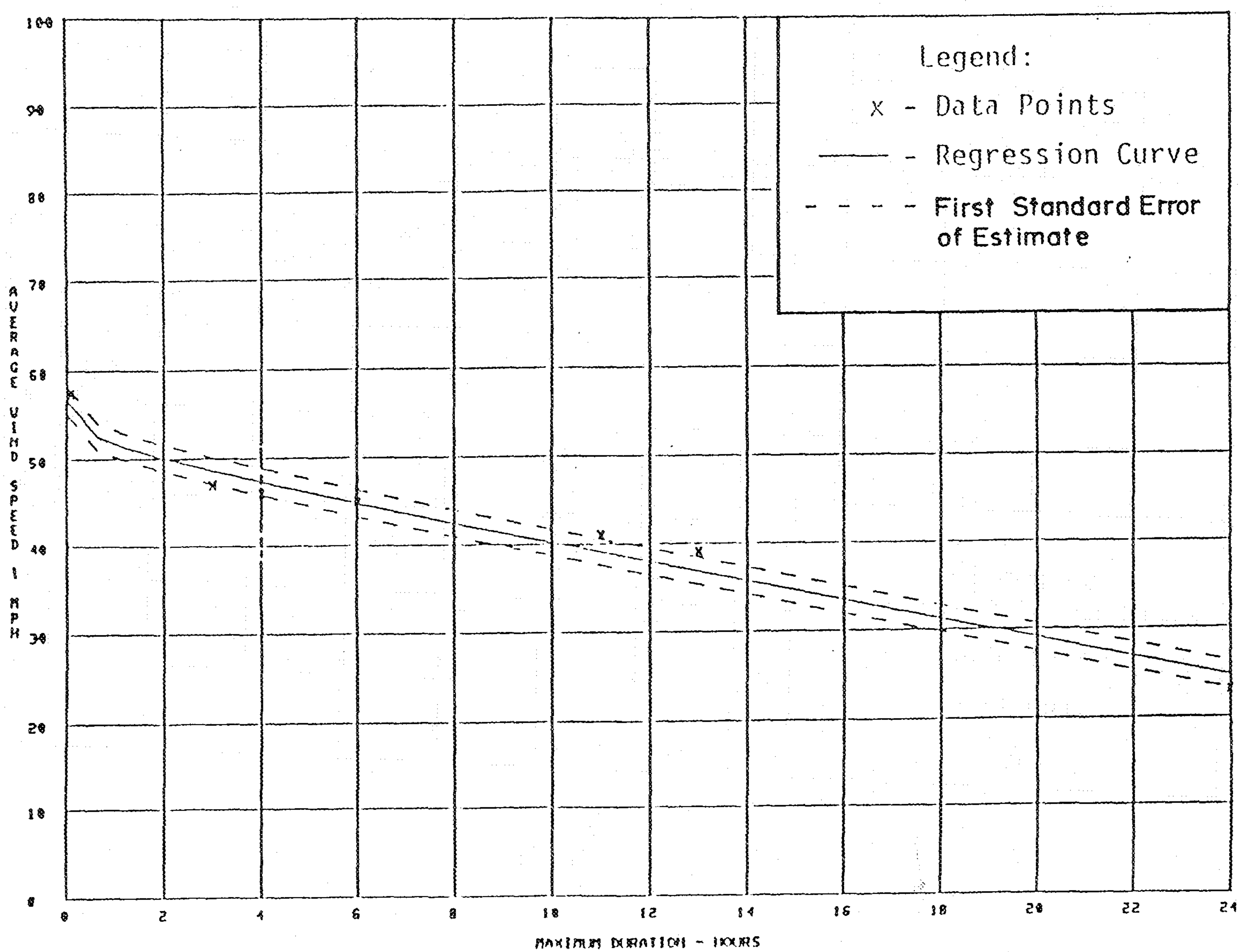
Northeast



WIND PERSISTENCE
MEASURED AT BOSTON MA
HYD.& W.Q. SECT. JUNE 1981

FIGURE B-5a

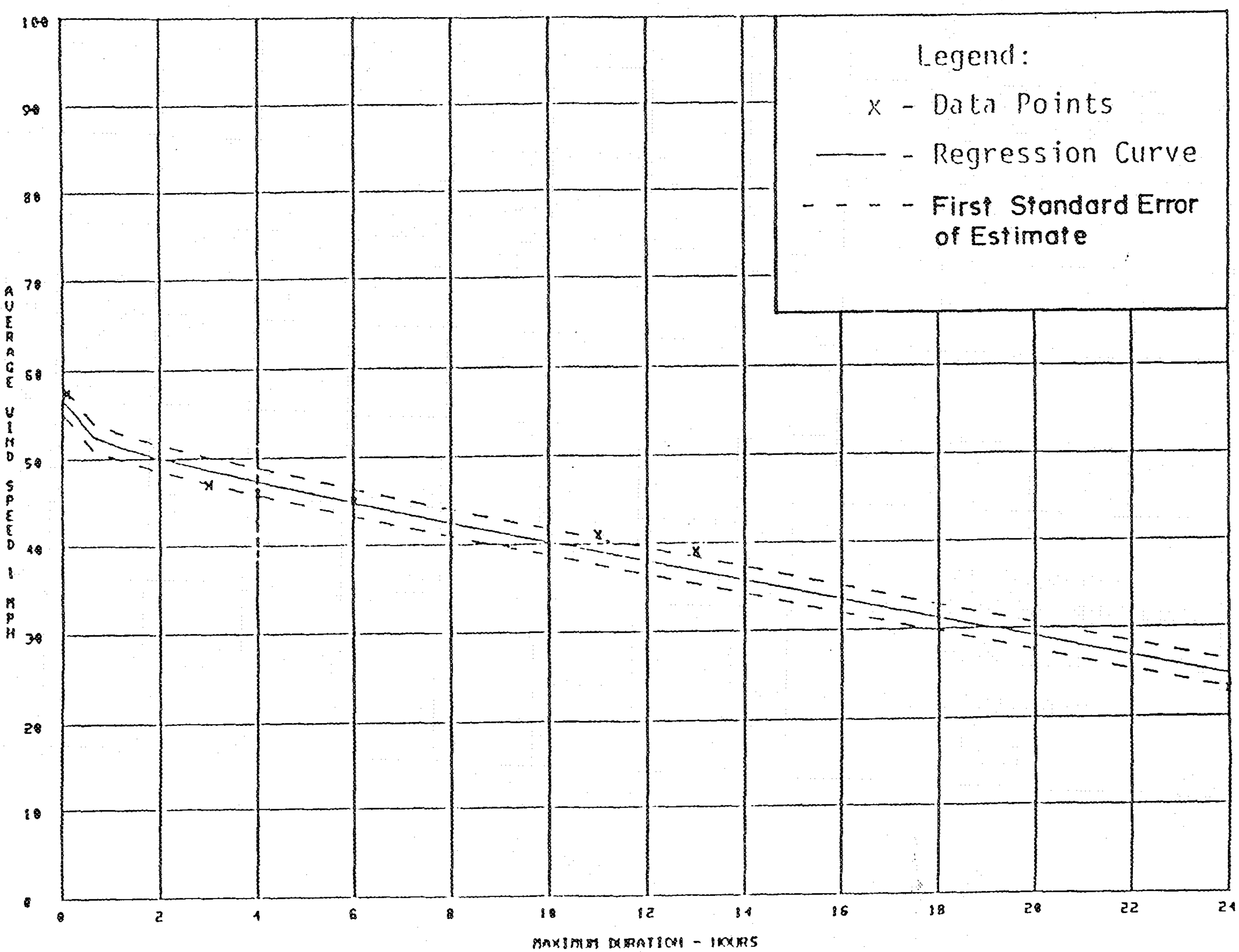
East-Northeast



WIND PERSISTENCE
MEASURED AT BOSTON, MA
HYD. & W.Q. SECT. JUNE 198?

FIGURE B-5b

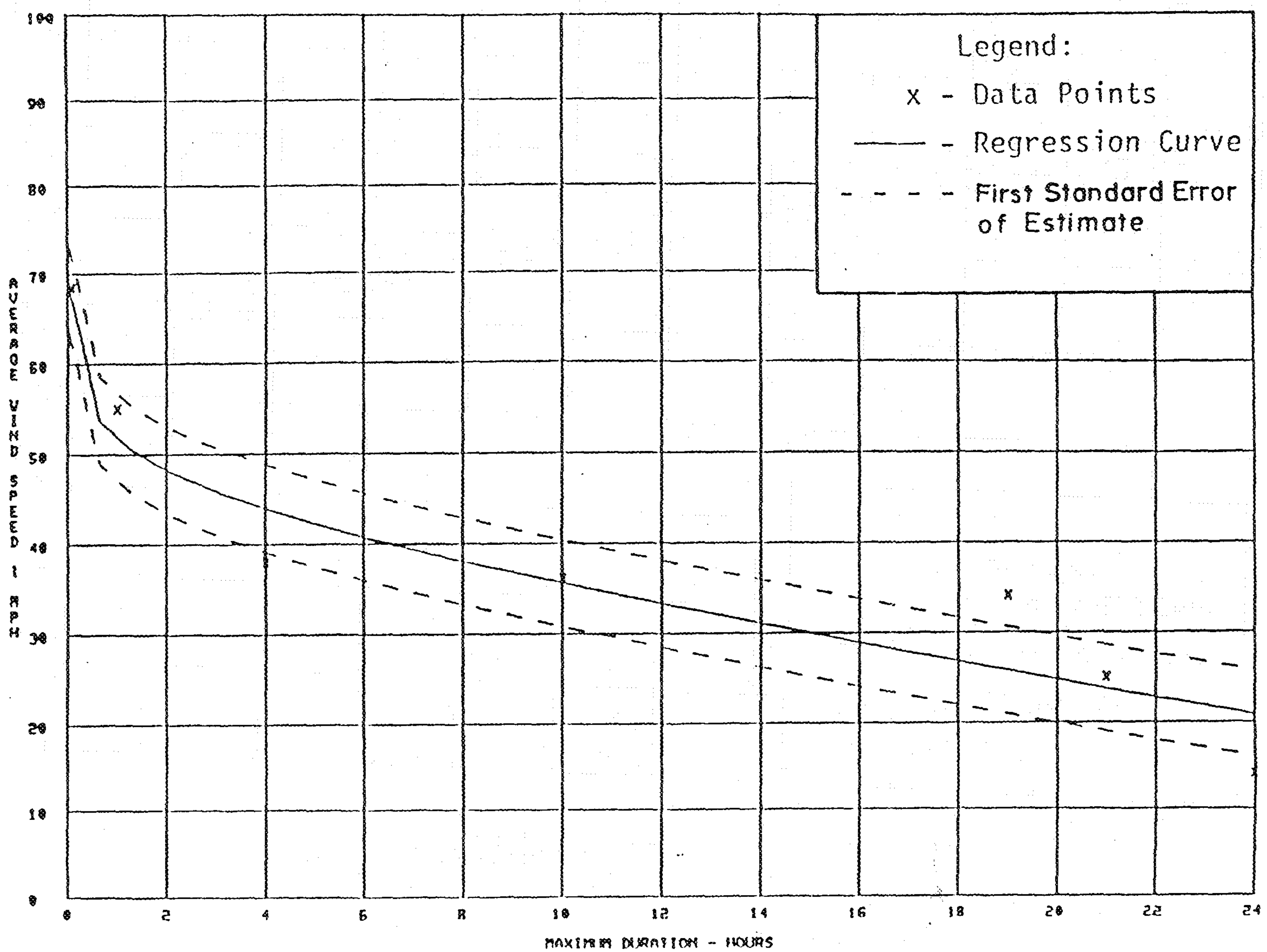
East-Northeast



WIND PERSISTENCE
MEASURED AT BOSTON, MA
HYD. & W.Q. SECT. JUNE 198

FIGURE B-5b

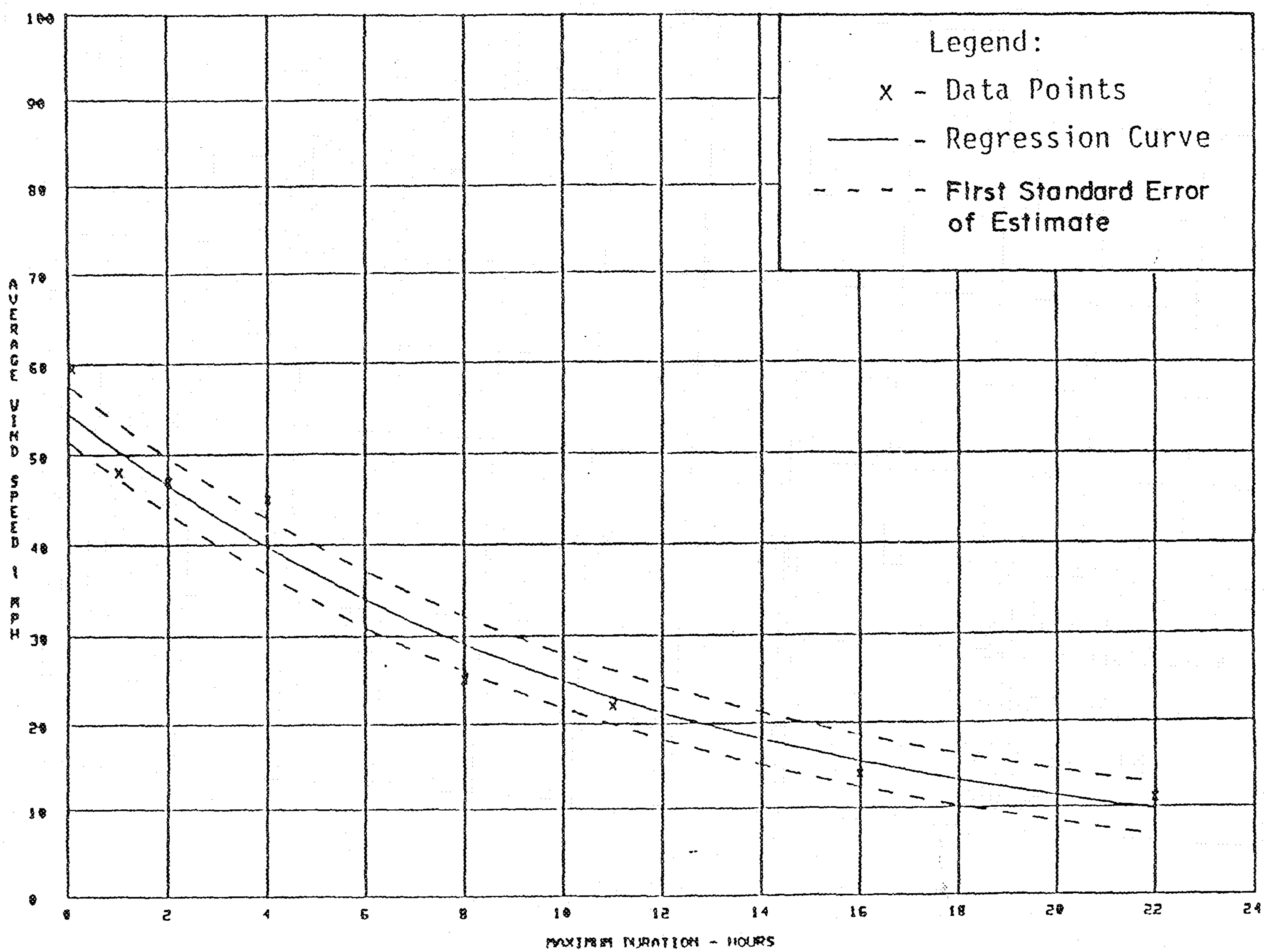
East



WIND PERSISTENCE
MEASURED AT BOSTON, MA.
HYD. & W.Q. SECT. JUNE 1988

FIGURE B-5C

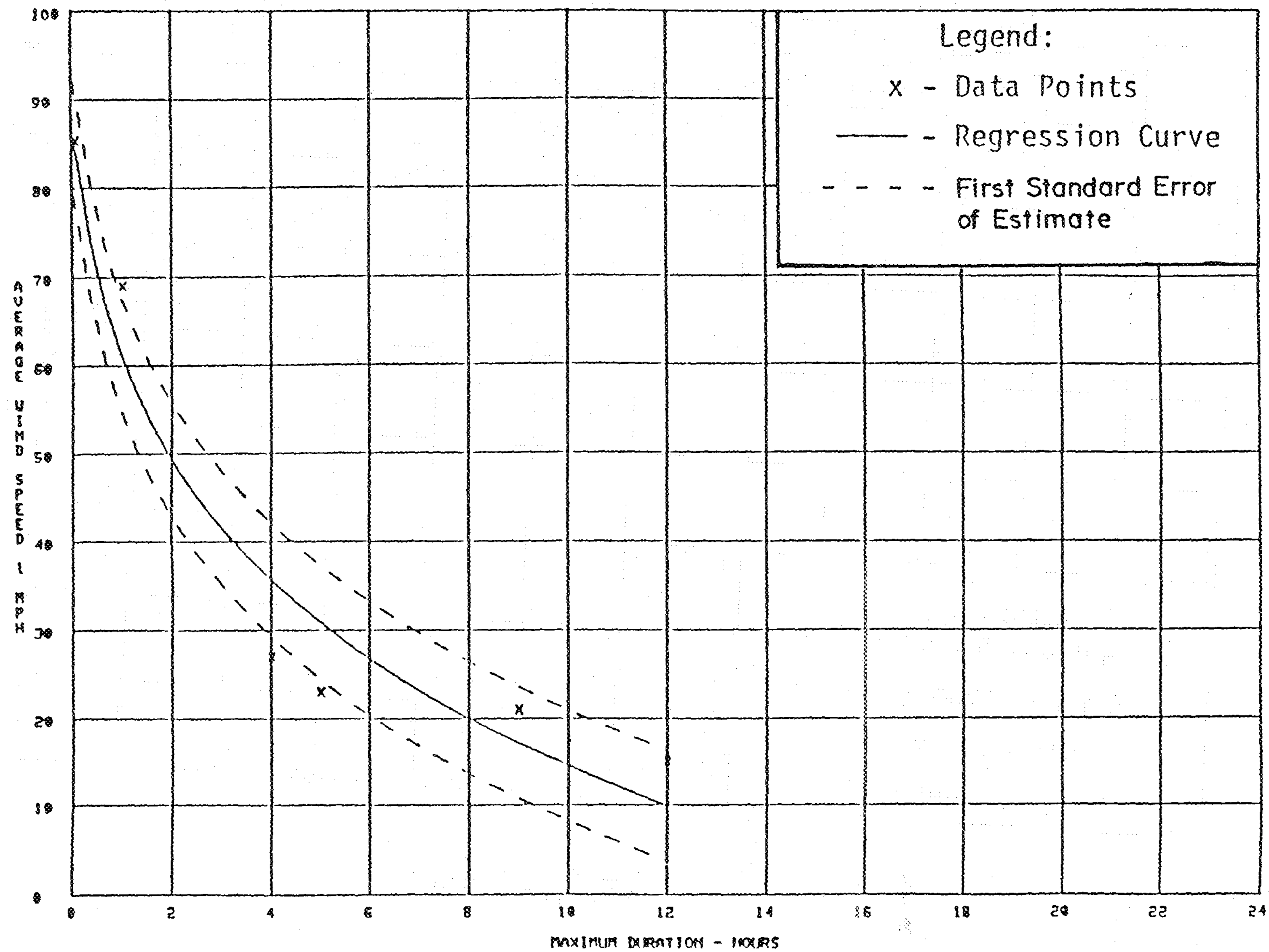
East-Southeast



WIND PERSISTENCE
MEASURED AT BOSTON, MA
HYD. & W.Q. SECT. JUNE 198

FIGURE B-5d

Southeast



WIND PERSISTENCE
MEASURED AT BOSTON, M.
HYD. & W.Q. SECT. JUNE 19

FIGURE B-5e

TABLE B-5

BOSTON - LOGAN INTERNATIONAL AIRPORT
NATIONAL WEATHER SERVICE
WIND OBSERVATIONS RECORDED
DURING NOTABLE TIDAL FLOODS

Date	Resultant Direction	Resultant Speed (mph)	Average Speed (mph)	Fastest-Mile Direction	Fastest-Mile Speed (mph)
06 Feb 1978	ENE	28.4	29.3	NE	61
29 Dec 1959	NE*	--	20.7	E	34
02 Jan 1987	N	15.5	21.8	NE	35
25 Jan 1979	ENE	23.2	24.2	E	45
30 Oct 1991	NNE	25.8	26.5	NE	37
19 Feb 1972	NE	21.1	24.2	NE	47
21 Apr 1940	-	--	13.3	NE	43**
12 Dec 1992	NE	31.8	32.1	NE	51
30 Nov 1944	-	--	13.4	NE	48**
25 May 1967	NE	34.3	34.7	NE	50
20 Jan 1961	NNW*	--	26.7	NNE	41
16 Mar 1956	ENE*	--	28.1	NE	54
06 Apr 1958	WSW*	--	13.8	SSE	32
16 Mar 1976	ENE	15.4	20.4	NE	35
07 Mar 1962	NE*	--	31.6	ENE	42
26 Feb 1979	NE	19.1	19.6	NE	30
02 Dec 1974	ENE	15.7	20.7	E	38
27 Feb 1952	N	--	17.4	NE	40

* Resultant speed and direction not available for the period prior to 1964; direction shown is prevailing wind direction.

** Fastest-mile not available; volume shown is five-minute average speed.

NOTE: Listing is in order of decreasing observed stillwater tide level to provide uniformity with table B-8

TABLE B-6

BOSTON - LOGAN INTERNATIONAL AIRPORT
NATIONAL WEATHER SERVICE
WIND OBSERVATIONS RECORDED
DURING ANNUAL MAXIMUM SURGE
PRODUCING STORMS
(1922-1979)

Date	Fastest Mile Speed (mph)	Fastest Mile Direction	Average Speed (mph)	Prevailing Direction
29 Nov 1945	63*	NE	40.5	-
13 Apr 1961	42	ENE	25.0	NE
06 Feb 1978	61	NE	29.3	ENE
14 Feb 1940	51*	NE	12.7	-
17 Nov 1935	54*	NE	14.9	-
19 Feb 1972	47	NE	24.2	NE
03 Mar 1947	50*	E	13.4	-
04 Mar 1960	45	NE	28.0	N
30 Jan 1966	43	S	22.3	SSE
12 Nov 1968	54	NE	23.9	E
25 Jan 1979	45	E	24.2	ENE
22 Mar 1977	60	NE	19.3	E
25 Nov 1950	74	E	42.4	E
31 Aug 1954	86	SE	31.8	ENE
16 Feb 1958	45	E	28.0	E
15 Nov 1962	37	NW	28.5	NW
16 Mar 1956	54	NE	28.1	ENE
27 Dec 1969	26	E	17.3	WNW
11 Mar 1924	--	-	--	-
30 Jan 1939	43*	NE	12.7	-
17 Feb 1952	50	NE	29.8	NE
07 Mar 1923	--	-	--	-
20 Feb 1927	--	-	--	-
19 Jan 1936	40*	NE	12.6	-
07 Nov 1953	67	NE	30.5	NE
14 Aug 1971	18	E	9.6	E
28 Jan 1973	23	NE	19.4	NE
12 Mar 1959	42	ESE	23.9	SE
16 Apr 1929	--	-	--	-
08 Mar 1931	--	-	--	-

* Fastest-mile not available; value shown is five-minute average speed.

NOTE: Listing in order of decreasing annual maximum storm surge to allow comparison with Table B-7

TABLE B-7

ANNUAL MAXIMUM STORM SURGE
BOSTON, MASSACHUSETTS
(1922-1979)

Date	Annual Maximum Storm Surge (feet)	Maximum Observed Tide Level for the Day (feet, NGVD)	Recurrence* Interval (years)
30 Nov 1945	4.9	7.6	LT 1
13 Apr 1961	4.7	8.0	1
06 Feb 1978	4.6	10.0	50
14 Feb 1940	4.2	5.0	LT 1
17 Nov 1935	4.1	6.5	LT 1
19 Feb 1972	4.0	9.1	7
03 Mar 1947	3.8	7.2	LT 1
04 Mar 1960	3.8	8.1	2
30 Jan 1966	3.8	5.5	LT 1
12 Nov 1968	3.7	7.7	LT 1
25 Jan 1979	3.7	9.2	10
22 Mar 1977	3.6	5.3	LT 1
25 Nov 1950	3.6	6.4	LT 1
31 Aug 1954	3.5	8.2	2
16 Feb 1958	3.5	7.9	1
15 Nov 1962	3.5	7.9	1
16 Mar 1956	3.4	5.6	LT 1
27 Dec 1969	3.3	6.7	LT 1
11 Mar 1924	3.2	6.2	LT 1
31 Jan 1939	3.2	6.9	LT 1
18 Feb 1952	3.2	7.9	1
07 Mar 1923	3.1	6.9	LT 1
20 Feb 1927	3.1	6.9	LT 1
19 Jan 1936	3.1	5.9	LT 1
07 Nov 1953	3.0	7.4	LT 1
14 Aug 1971	3.0	5.4	LT 1
29 Jan 1973	3.0	6.1	LT 1
12 Mar 1959	2.9	6.5	LT 1
16 Apr 1929	2.8	6.6	LT 1
08 Mar 1931	2.8	6.5	LT 1

* Recurrence interval of observed tide elevations. Obtained from tide stage-frequency relationship, Figure B-8

TABLE B-8

MAXIMUM STILLWATER TIDE HEIGHTS
BOSTON, MASSACHUSETTS

Date	Observed Elevation (feet, NGVD)	Adjusted Elevation* (feet, NGVD)	Weibul Plotting Position*** (percent)
26 Dec 1909	9.9	10.6	0.7
16 Apr 1851	10.1	10.5	1.5
07 Feb 1978	10.4	10.5	2.5
29 Dec 1959	9.4	9.7	3.5
02 Jan 1987	9.5	9.5	4.5
25 Jan 1979	9.3	9.5	5.5
30 Oct 1991	9.4	9.5	6.5
19 Feb 1972	9.2	9.4	7.4
21 Apr 1940	8.9	9.4	8.2
12 Dec 1992	9.4	9.4	9.4
29 Dec 1853**	8.9	9.3	10.4
04 Mar 1931	8.8	9.3	11.4
03 Dec 1854**	8.8	9.3	12.4
30 Nov 1944	8.8	9.2	13.3
26 May 1967	9.0	9.2	14.3
03 Nov 1861**	8.7	9.2	15.7
20 Jan 1961	8.9	9.1	16.7
15 Nov 1871**	8.5	9.1	17.7
23 Nov 1858**	8.5	9.0	18.6
17 Mar 1956	8.6	9.0	19.6
07 Apr 1958	8.6	8.9	20.6
16 Mar 1976	8.7	8.9	21.6
28 Jan 1933	8.3	8.8	22.6
31 Dec 1857**	8.3	8.8	23.5
07 Mar 1962	8.5	8.7	24.5
06 Jan 1856**	8.2	8.7	25.5
09 Jan 1868**	8.1	8.6	26.5
21 Nov 1851**	8.2	8.6	27.5
07 Mar 1864**	8.1	8.6	28.4
26 Feb 1979	8.5	8.6	29.4
02 Dec 1974	8.4	8.6	30.4
12 Nov 1947	8.2	8.6	31.4

* Observed values after adjustment for changing mean sea level; adjustment made to 1992 mean sea level.
** Approximate values based on historical account.
*** Weibul plotting position of adjusted elevation with annual series/partial duration adjustment for 101 years of record, 1848-1992 (1 high outlier).

NOTE: Events occurring within about 30 days of a greater tide producing event are exluded from this list. Events recorded during years for which only partial records are available were also excluded.

corrections by NOS in some data records). The recent 30 October 1991 storm had a storm surge of 5.1 feet, greater than any shown in table B-7.

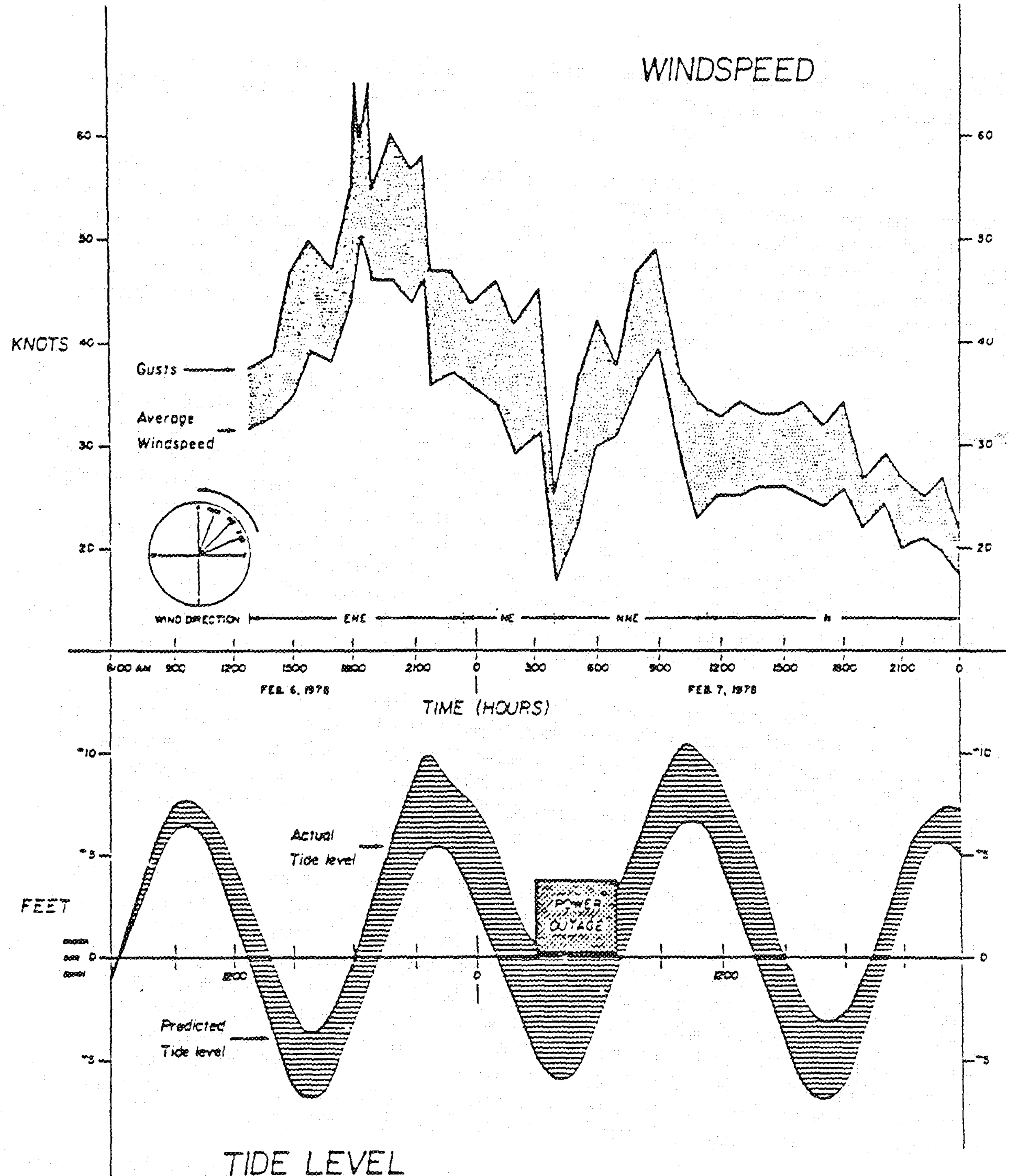
Some of the most severe onshore winds, waves, and storm surges have produced minor tidal flooding, due to their coincidence with low astronomic tides. An example of this is the 30 November 1945 event, producing near record storm surge at Boston; extremely high onshore winds occurred during a low astronomic tide, resulting in only a minor stillwater tidal flood level (7.6 feet NGVD).

Conversely, rather significant tidal flood levels can result from the coincidence of relatively high astronomic tides and relatively minor meteorological events. Astronomic high tide level in Boston alone can reach 7.5 feet NGVD (see table B-3). With such a condition, a coincident storm surge of only 2 to 3 feet can produce major tidal flood levels. The 7 February 1978 storm tide at Boston reached 10.4 feet NGVD, the greatest of record, but was produced by an astronomic tide of 6.9 feet NGVD and surge of 3.5 feet combination; the latter being of only moderate magnitude (see table B-7, which shows a surge of 3.5 feet is not extreme).

Windspeed observations recorded by the NWS at Boston's Logan Airport during the great blizzard of '78 are shown on figure B-6. There were gusts from ENE in excess of 55 knots (63 mph) for approximately four hours. Average windspeeds from the same direction were sustained above 43 knots (49 mph) for nearly four hours.

(4) Study Area. Winds from ENE were used at Short Beach. Information was obtained from the above wind analysis at Logan Airport, initially developed for Revere. Analyses at Black Rock Beach were based on winds from the WNW and WSW, the only directions yielding wave generation in the area. Windspeed and duration were estimated by Coastal Engineering Branch using table D-2 of the 1968 Nantasket Beach Report. Winds from these directions do not affect Revere Beach and were not presented in section 4c, regarding winds during storms producing surge in the area.

d. Storm Tides. The total effects of astronomical tide combined with storm surge produced by wind, wave, and atmospheric pressure contributions are reflected in actual tide gage measurements. Since the astronomical tide is so variable at the study area, time of occurrence of the storm surge greatly affects the magnitude of the resulting tidal flood level. Obviously, a 3-foot storm surge, occurring at a low astronomical tide, would not produce as high a water level as one occurring at a higher tide. It is important to



REPRINTED FROM:

METROPOLITAN DISTRICT COMMISSION
ENVIRONMENTAL IMPACT REPORT
FOR THE
REVERE BEACH DEVELOPMENT PROJECT

TIDE LEVEL AND WINDSPEED VS. TIME FOR THE STORM OF FEBRUARY 6 AND 7, 1978.

Comp, Dresser & McKee, Inc.

Alan M. Voorhees & Assoc., Inc. Bolt Beranek & Newman, Inc.

FIGURE B-6

note the storm surge itself varies with time, thus introducing another variable into the makeup of the total flood tide.

(1) Boston. The variation in observed tide and surge at Boston, during the "Blizzard of '78," is shown in figure B-7. It is interesting to note the maximum surge (4.7 feet) occurred just before 10 p.m. on 6 February. However, the maximum observed tide occurred about 10:30 a.m. the following day when the surge had dropped by 1.2 feet. Had the maximum surge recorded during the storm occurred at 10:30 a.m. on 7 February, the observed tide would have been 11.6 feet NGVD, and would have resulted in even more catastrophic flooding at the project area. Annual maximum surge values of greater than or equal to 3.0 feet, measured at the Boston, Massachusetts, National Ocean Survey (NOS) tide gage are shown on table B-7. The average annual maximum storm surge at Boston is 3 feet. This table shows the importance of coincident astronomic tide in producing significant tidal flooding (see the discussion in section 4c, which deals with wind observations recorded during these events).

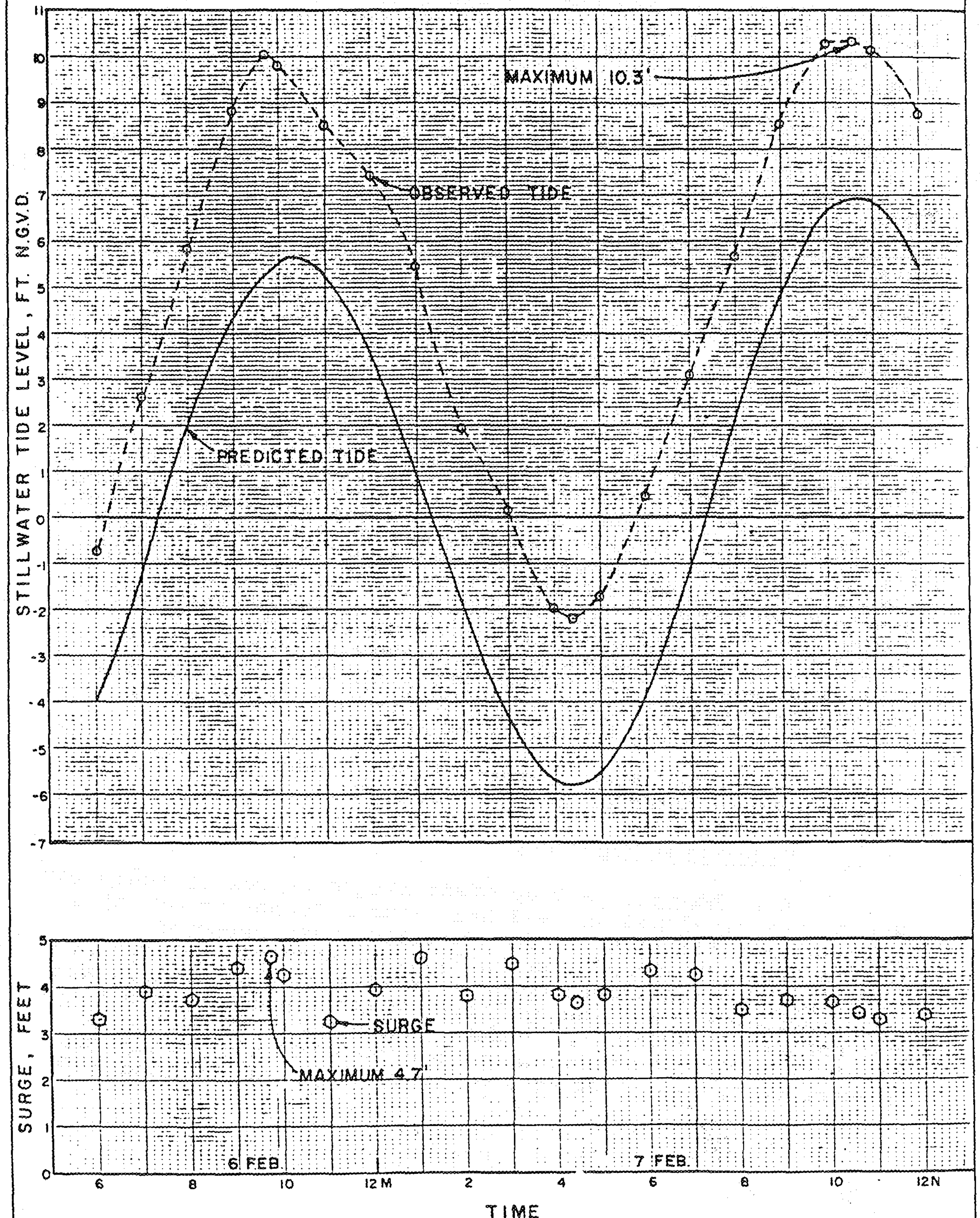
The NOS has systematically recorded tide heights at Boston since 1922. The record prior to that time was developed by utilizing staff gage measurements and historical accounts. Maximum observed stillwater tide heights (measurements taken in protected areas where waves are dampened out), recorded through 1992, are shown in table B-8. Also shown are tide heights, with an adjustment applied to account for the effect of rising sea level (see section 14). The greatest observed stillwater tide level recorded occurred during the "Great Blizzard of '78." No hurricanes or tropical storms are known to have produced extreme tidal heights at Boston, thus indicating that, historically, the principal threat of flooding in the study area is due to storms of the extratropical variety.

(2) Study Area. Storm tides at Nahant are very close to those observed at the Boston NOS gage.

e. Tide-Stage Frequency

(1) Boston. A tide stage-frequency relationship for Boston was developed in 1992 utilizing a composite of (a) a Pearson type III distribution function, with expected probability adjustment, for analysis of historic and systematically observed annual maximum stillwater tide levels, and (b) a graphical solution utilizing Weibull plotting positions for partial duration series data (reference: EM 1110-2-1412, 15 April 1986). Corrections to previous NOS tide data, the occurrence of two major coastal storms, and the release of a

"BLIZZARD OF '78"   6-7 FEBRUARY 1978
BOSTON, MASS.



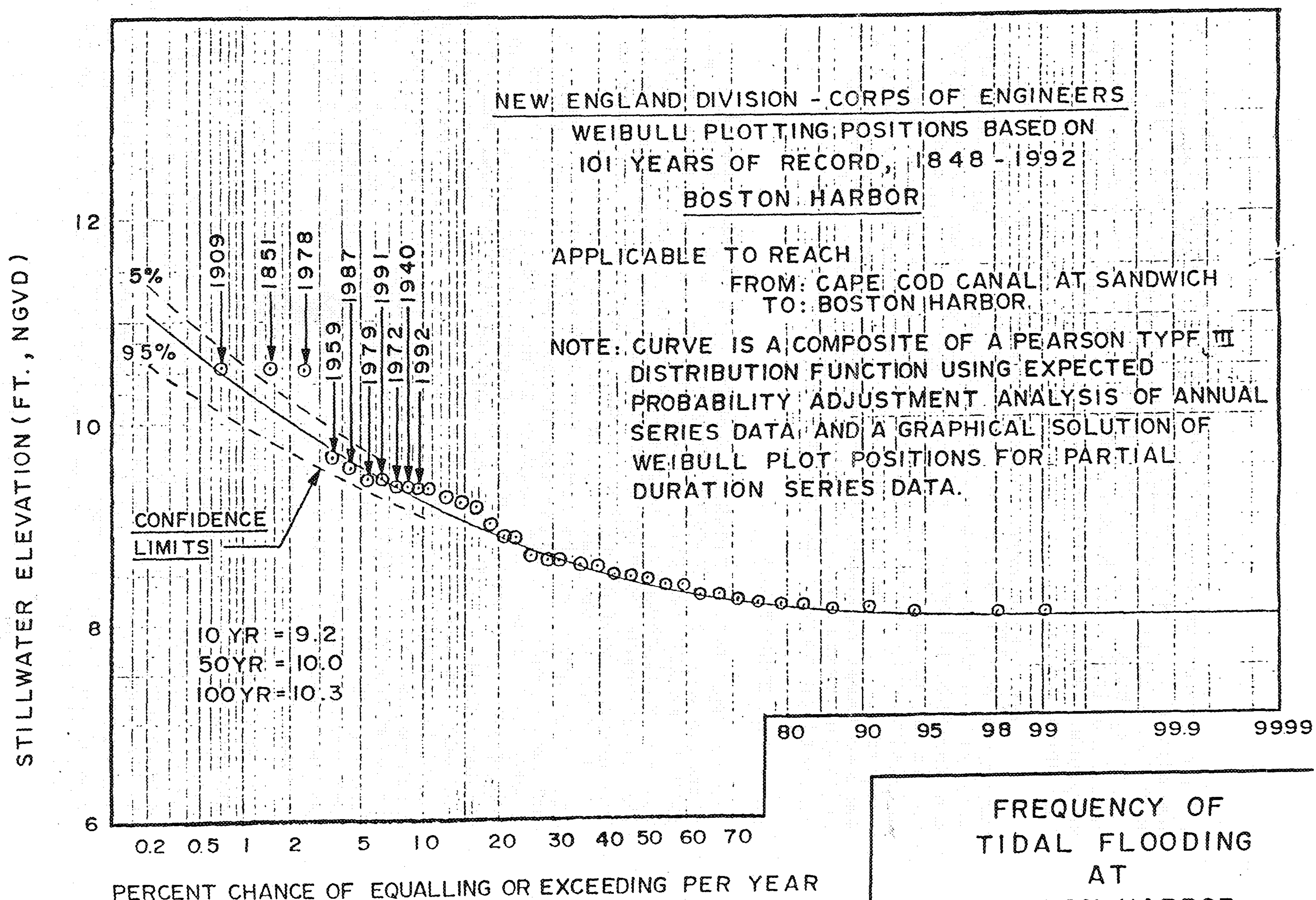
new NOS report on sea level rise in 1988, prompted this reanalysis from previous studies in 1979. Due to greater confidence in sea level rise, estimated since 1922, the recent systematic curve was adopted. However, plotting positions including historic data were used for descriptive purposes. The resulting tide stage-frequency curve is shown on figure B-8.

(2) Study Area. NOS tide gage records and high watermark data have been gathered, after major storms were utilized in the development of profiles of tidal floods along the New England coast. Additionally, profiles of storm tides for selected recurrence intervals have been developed, utilizing tide stage-frequency curves and high watermark information. A location map and profile for the reach of New England coast bounding the project are shown in figures B-9 and B-10, respectively. These generally show stillwater storm tides to be similar between Boston and Nahant.

5. TIDAL HYDRAULICS

a. Standard Project Northeaster (SPN) Tide Level. Although wave overtopping was not conducted for the SPN at Nahant, an SPN has been defined for an adjacent area at Revere, Massachusetts, and is described as follows: Previous analysis conducted during the feasibility investigation for the nearby Roughans Point project in Revere, Massachusetts, resulted in an estimated ocean stillwater tide level of 13.0 feet NGVD for the SPN. OCE approved use of this estimated value, pending formal development of the SPN tide level (reference: DAEN-CWE-H, 17 November 1980, 1st Ind, "Hydrologic Criteria - Revere, Massachusetts Coastal Flood Protection"). During a subsequent meeting between NED, WES, and OCE, it was agreed that a less formal analysis of the SPN would be conducted by WES for use in the Revere area, along with physical and mathematical modelling of wave overtopping at Roughans Point (reference: DAEN-CWH-Y, 5 March 1984, 1st Ind and NED-ED-WQ, 11 April 1984, 2nd Ind, "Hydrologic Criteria - Roughans Point Coastal Flood Protection, Revere, Massachusetts"). The following presents a discussion on WES evaluation of the Standard Project Northeaster Tide Level.

The Standard Project Northeaster definition can be determined from the definition for the Standard Project Storm (EM 1110-2-1411) as the northeaster resulting from the "most severe combinations of meteorological and tidal conditions considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations." For this report, two processes are important in considering the specification of an SPN stillwater level and wave overtopping. It is possible that a separate SPN would have to be



FREQUENCY OF
TIDAL FLOODING
AT
BOSTON HARBOR

CJW MAR. 9

FIGURE B-8

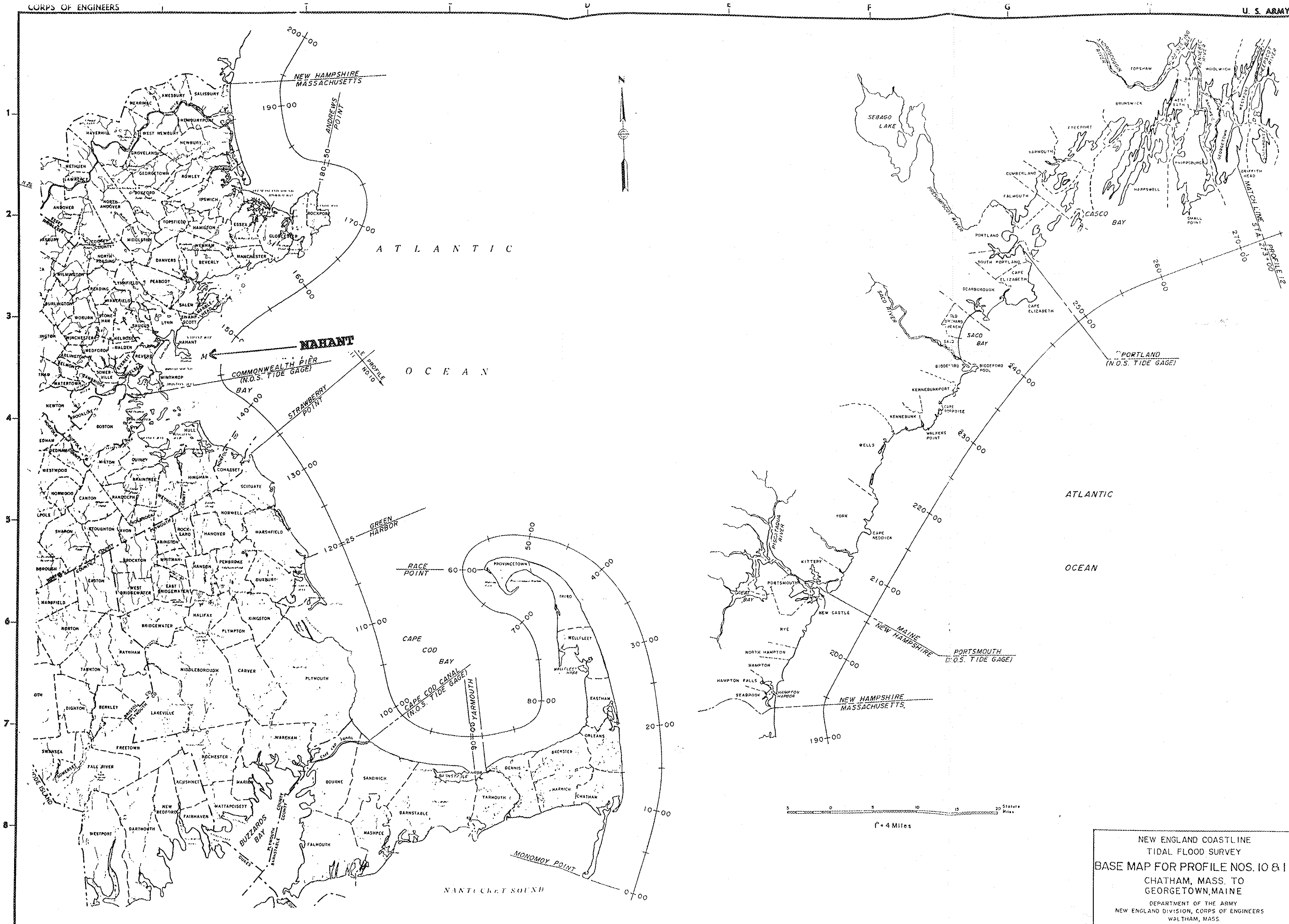
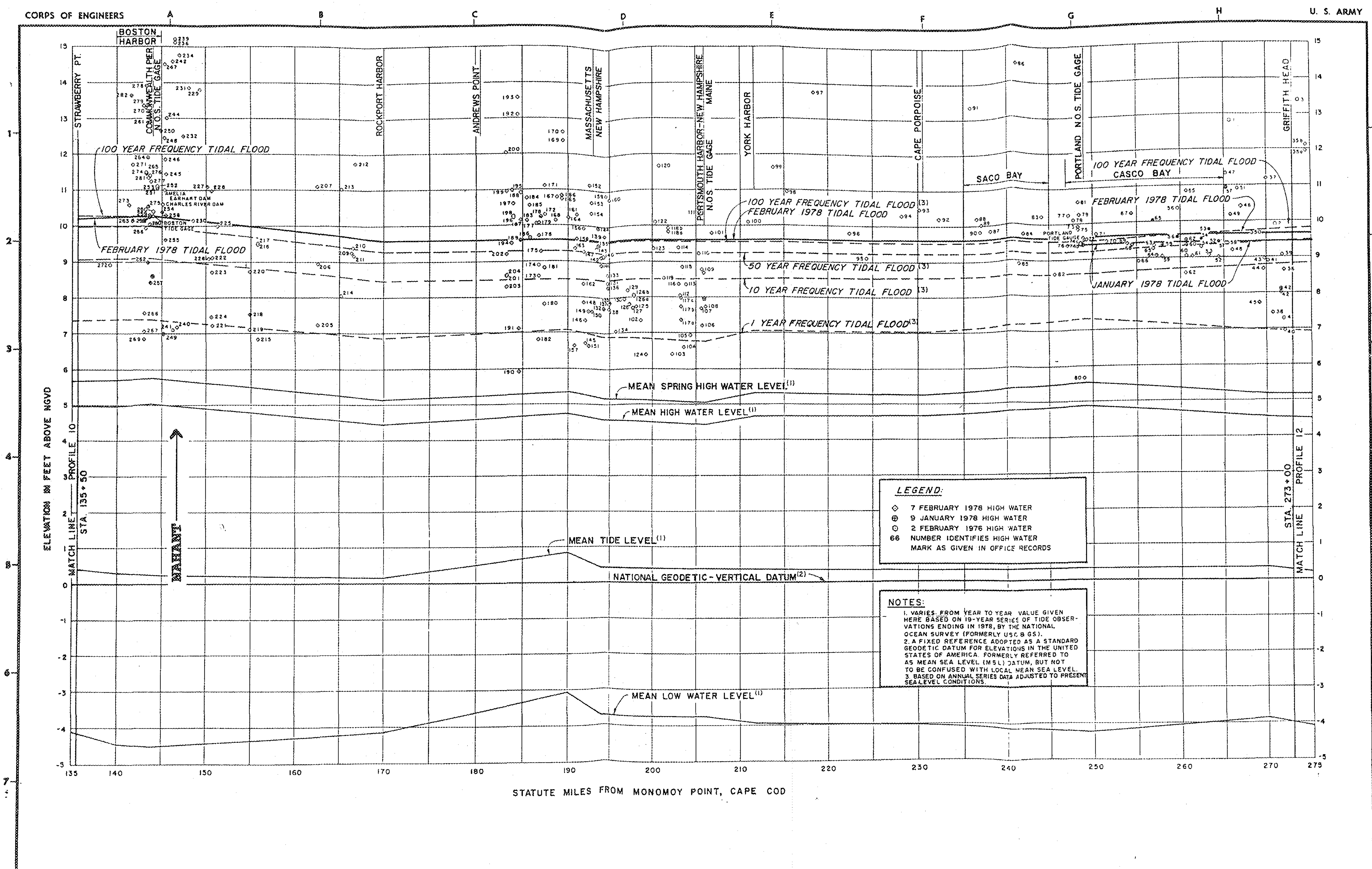

CORPS OF ENGINEERS
U. S. ARMY
ATLANTIC
OCEAN
NAHANT
NEW HAMPSHIRE
MASSACHUSETTS
ANDREWS POINT
COMMONWEALTH PIER (N.O.S. TIDE GAGE)
STRAWBERRY POINT
GREEN HARBOR
RACE POINT
CAPE COD BAY
CAPE COD CANAL (N.O.S. TIDE GAGE)
YARMOUTH
MONOMOY POINT
NANTUCKET SOUND
BUZZARDS BAY
SEBAGO LAKE
CASCO BAY
SACO BAY
PORTLAND (N.O.S. TIDE GAGE)
MAINE
NEW HAMPSHIRE
PORTSMOUTH (N.O.S. TIDE GAGE)
NEW HAMPSHIRE
MASSACHUSETTS
1" = 4 Miles
NEW ENGLAND COASTLINE
TIDAL FLOOD SURVEY
BASE MAP FOR PROFILE NOS. 10 & 11
CHATHAM, MASS. TO
GEORGETOWN, MAINE
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.

FIGURE B-9



NEW ENGLAND COASTLINE
TIDAL FLOOD SURVEY

TIDAL FLOOD PROFILE NO. 11

COHASSET, MASS. TO
GEORGETOWN, MAINE

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.
SEPTEMBER 1988

FIGURE B-10

defined for each process. The SPN which would produce the highest ocean stillwater level might not produce the highest waves in the study area and, therefore, not the highest overtopping rates.

The SPN stillwater level was estimated to be 13.0 feet NGVD in NED's feasibility studies for Revere, by adding together the maximum surge recorded at Boston, approximately 5 feet, and the maximum predicted astronomic tide, 7.5 feet NGVD, and then rounding up to the next foot of elevation.

This resulted in a stillwater ocean elevation almost 3 feet higher than the maximum ever recorded at the Boston gage. Given the unlikely event that a tide, with a maximum elevation near the maximum predicted astronomic tide, were to occur sometime during the maximum surge-producing north-easter, the probability that the hour of maximum surge (using hourly increments) would occur at the hour of maximum tide is only 1/24 (assuming a semidiurnal tide with unequal highs). Consequently, this combination might fall under the "excluding extremely rare" clause in the definition of the SPN. A better specification of the SPN stillwater level might be closer to 12.0 feet NGVD. Because of the close proximity of Revere to Nahant, for purpose of this study, a revised SPN stillwater tide level of 12 feet NGVD has been adopted.

b. Wave Height, Runup and Overtopping. As part of the reconnaissance investigation for flood protection of Nahant's Black Rock and Short Beaches and backshore area, wave heights, wave runup, and peak rates of overtopping were computed.

(1) Existing Conditions. Typical beach and seawall profiles used for wave runup and overtopping calculations for Black Rock and Short Beaches were surveyed by the Coastal Engineering Branch on 24 June 1994 and 16 August 1994. The Black Rock Beach area consists of Town Beach, several houses fronted by natural slopes, and several houses fronted by privately-owned seawalls. Town Beach has a crest elevation of 9.8 feet NGVD, and an estimated slope of 1 vertical to 10 horizontal. Seawalls front approximately 65 percent of the houses along Black Rock Beach, and range in elevation from 10.3 to 13 feet NGVD. Some walls are fronted by rock revetments, providing additional protection from undermining. A near shore slope for Black Rock Beach of 1 vertical to 100 horizontal was assumed.

Short Beach, on the east side of Nahant, has a crest elevation ranging from approximately 13 to 14 feet NGVD. A beach slope of 1 vertical to 30 horizontal was estimated from surveys and a near shore slope of 1 vertical to 100 horizontal was assumed.

(2) Design Wave Heights, Periods and Runup. Wave heights, periods, and runup were computed by the Coastal Engineering Branch for overtopping analyses, using the Automated Coastal Engineering System (ACES). At Black Rock Beach, the analysis was based partly on 2-mile limited fetch lengths for the WSW direction and 1 mile for WNW direction. Analyses performed showed that conditions under WSW winds were more severe than WNW; therefore, WSW winds were used along Black Rock Beach for overtopping calculations. A fetch length of 275 miles in the ENE direction was used at Short Beach. Results of wave runup calculations, for particular return periods and existing conditions, are shown in tables B-9 and B-10. Wave heights calculated for Black Rock Beach are small due to limited fetch lengths and shallow water. The crest elevations estimated to prevent overtopping on Black Rock Beach are considered conservative, since stillwater levels used are more typically associated with storm winds from the northeast through southeast directions. Such storms include a storm surge component in the stillwater level for a given frequency, which could be missing during storms with westerly winds.

(3) Proposed Conditions. Proposed conditions, determined by Planning Directorate from runup analyses, initially consisted of either a seawall along Black Rock Beach at 12.5 or 13 feet NGVD, or a beach at elevation 11 feet NGVD. The beach would front the areas which currently are unprotected, while the wall would be continuous along the entire length of the reach.

Runup analyses on Short Beach indicated that overtopping did not occur until the area experienced storms having between 50 and 100-year return periods. It was determined that protective measures would not be cost effective along Short Beach, and there would be no proposed conditions for overtopping analysis.

(4) Analysis of Peak Overtopping Rate

(a) General. The Automated Coastal Engineering System (ACES), version 1.07e "Wave Runup and Overtopping on Impermeable Structures" was used to estimate peak rates of overtopping along existing seawalls and beaches. For each particular return period, a local windspeed from the west-southwest (Black Rock Beach) and east-northeast (Short Beach) direction was assumed to be occurring during the overtopping

TABLE B-9

WAVE HEIGHT, PERIOD AND RUNUP
BLACK ROCK BEACH

Freq (yrs)	SWL (ft, NGVD)	Wind Speed (mph)	Wave Height (ft)	T (sec)	Runup Beach (ft)	Runup Wall (ft)	Req'd Wall Elev* (ft,NGVD)
2	8.1	15	0.8	1.7	0.3	1.7	9.9
5	8.7	16	0.9	1.8	0.4	1.9	10.6
10	9.1	18	1.0	1.8	0.4	2.0	11.1
25	9.6	19	1.0	1.9	0.4	2.1	11.7
50	10.0	20	1.1	1.9	0.5	2.3	12.3
100	10.3	22	1.3	1.9	0.5	2.6	12.9

- wave height from Rayleigh distribution for shallow water
- average depth used = 15 feet
- beach slope 1:10, wall slope 1:2
- fetch = 2 miles, winds from WSW

* required elevation to prevent overtopping at each frequency

TABLE B-10

WAVE HEIGHT, PERIOD AND RUNUP
SHORT BEACH

Frequency (yrs)	SWL (ft,NGVD)	Wind Speed (mph)	Wave Height (ft)	T (sec)	Runup (ft)
2	8.1	23	6.9	8.5	1.7
5	8.7	27.5	7.4	10.2	2.1
10	9.1	31.5	7.8	11.7	2.5
25	9.6	35	8.3	13.0	2.8
50	10.0	39	8.7	14.4	3.2
100	10.3	42.5	9.0	15.7	3.6

- beach slope 1:30
- fetch = 275 miles, fully developed sea
- winds from ENE

period. Overtopping coefficients were estimated, using the 1984 Shore Protection Manual, and best engineering judgement.

The condition when waves break at the structure toe was assumed to be critical, producing the maximum wave runup and peak rates of overtopping of the existing conditions. The structure toe is defined at the base of the seawall, or the point where the structure slope intersects the nearshore slope. Depth at the structure toe was assumed to be the difference between tidal stillwater level for the particular return period and elevation of the intersection of the structure and nearshore slope.

(b) Black Rock Beach. Table B-11 presents results of the overtopping analysis along Black Rock Beach for both existing and proposed conditions. A seawall at 12.5 feet NGVD will reduce overtopping to negligible amounts for events having less than a 100-year return period. A 13-foot NGVD seawall would reduce overtopping to a negligible

TABLE B-11						
BLACK ROCK BEACH OVERTOPPING ANALYSIS						
		EXISTING CONDITIONS			PROPOSED CONDITIONS	
AREA DESCRIPTION		TOWN BEACH & HOUSES WITH NO WALLS	63,55, 53,49 CASTLE ROAD - VERT WALLS	29-45 CASTLE ROAD - SLOPED REVET-MENTS	VERT WALL	VERT WALL
TOP OF WALL FT, NGVD		9.8	10.3-12	12-13	12.5	13
REACH LENGTH (FT)		530	300	270	1100	1100
FREQ (YRS)	STILL-WATER LEVEL NGVD	MAX OVERTOPPING RATE PER REACH (CFS/LF)				
100	10.3	***	1.34	0.12	0.03	neg
50	10.0	***	0.81	0.024	neg	neg
25	9.60	0.220	0.4	0.00	neg	neg
10	9.10	0.00	0.17	0.00	neg	neg
5	8.70	0.00	0.04	0.00	neg	neg
2	8.20	0.00	0.00	0.00	neg	neg

* Frequency based on tidal still water elevation curve for Boston, MA, (see figure B-8).

** Volume of overtopping is reflected in stage-frequency curves presented in paragraph 6.

*** Crest of structure below stillwater level; therefore, overtopping calculations not performed amount for all events evaluated. The volumes of overtopping were used to aid in the development of stage-frequency curves presented in paragraph 6.

(c) <u>Short Beach</u>. Table B-12 presents results of the overtopping analysis along Short Beach under existing conditions. Results confirm the belief of area residents interviewed during field surveys that not much overtopping starts to occur at Short Beach until they experience a relatively rare event.

(5) <u>Conclusions</u>. Existing conditions at Black Rock Beach result in flooding of the backshore area. A seawall at 13 feet NGVD would essentially eliminate substantial overtopping up to an event having greater than a 100-year return period. A 12.5-foot NGVD seawall would reduce overtopping to negligible amounts for events having return periods less than 100 years.

Short Beach does not experience overtopping until a storm in excess of a 50-year event occurs, and, therefore, protective alternatives were not considered in this area.

6. INTERIOR HYDROLOGY

a. <u>Description of Area</u>. The area under study, the "Lowlands Park", is located in the northwestern corner of Nahant. Development consists mostly of single family homes. Street elevations vary between 4.3 in Ward Road near the Lowlands Park to 10.0 feet NGVD in Castle Road near Town Beach. The drainage area is approximately 0.1 square mile. Due to its location, during rare events, this area is flooded by wave overtopping from both Lynn and Nahant Harbors. Flooding of the Nahant Beach Parkway may possibly cause isolation of the town of Nahant, since this is the only access to the mainland. Four distinct reaches were identified for analysis purposes from Corps survey and site visits. Reaches 1 through 3 are located along Lynn Harbor and reach 4 along Nahant Harbor. A short description of each follows.

(1) **Reach 1** includes Town Beach (150 feet long) and several houses north of it that are open to the ocean, and lack any flood protection. Ground elevations of this 530-foot long reach were estimated between 8.6 and 9.5 feet NGVD.

(2) **Reach 2**, immediately north of reach 1, is approximately 300 feet long. Houses located in here have privately-owned seawalls, with top elevations between 10.3 and 12 feet NGVD.

TABLE B-12		
SHORT BEACH OVERTOPPING ANALYSIS		
BEACH CREST ELEVATION IN FEET NGVD		13.5 - average of 3 surveyed cross sections
LENGTH OF REACH (SHORT BEACH) FEET		800
FREQUENCY (YRS)	STILLWATER TIDE LEVEL (FT, NGVD)	OVERTOPPING RATE (CFS/LF)
100	10.3	1.78
50	10.0	neg
25	9.6	neg
10	9.1	neg
5	8.7	neg
2	8.2	neg

* Frequency based on tidal stillwater elevation curve for Boston, MA, (see figure B-8).

** Volume of overtopping is reflected in stage-frequency curves presented in paragraph 6.

(3) **Reach 3**, the last in the Lynn Harbor side, is approximately 270 feet long, and located immediately north of reach 2. Privately-owned seawalls here range between 12 and 13 feet NGVD. One house lacks a seawall or other protective measures. For analysis purposes, the ocean exposure of this property was added to reach 1.

(4) **Reach 4** is the section of Short Beach which contributes to flooding in the Lowlands Park. Wave overtopping through an approximate length of 800 feet in this reach occurs for major storms such as a 100 year event. Revetment elevation in this area ranges between 13.0 to 14.0 feet NGVD. Overtopping from this side of the study area discharges directly into Lowlands Park.

b. Drainage System. According to information provided by town officials, the area of study has a 36-inch drain pipe located along an easement section near the intersection of Castle and Ward Roads that drains floodwaters back into Lynn Harbor. This drain, however, is considered inoperable due to accumulation of silts and sand in the harbor side flap gate which impeded its function. Maintenance of this drainage pipe would improve drainage conditions of the area.

During major storms, when significant wave overtopping occurs, the drainage system is overloaded causing street flooding and local ponding at lower elevations, particularly along Ward and Fox Hill Roads. Floodwaters from the Lowlands Park are drained by a 2-to 3-foot wide trench/36-inch pipe combination excavated parallel to Ward Road that drains into the golf course at the Bear Pond area. Double 24-inch culverts, under Flash Road, provide for the drainage of the Lowlands Park into the golf course to the south. According to local residents, this ditch is the only drainage option to floodwaters and, due to lack of maintenance, it is not as wide as when originally excavated. Floodwaters at the golf course are drained by double 24-inch culverts and tidal gates into Pond Beach. Location of Lowland Park, Bear Pond, and golf course area are shown in plate B-2.

c. Recent Flood Events. The most recent storms experienced in the area were the 6-7 February 1978, 30-31 October 1991, and the lesser storm of 12 December 1992. The "Blizzard of 78" is one of the largest of recent history. A warm, moist, low level, low pressure system from the mid-Atlantic Ocean, combined with an intense, high level, very cold, low pressure mass from Canada. The result of this match was a storm with an estimated return frequency of a 100-year event, with storm induced tidal surges in excess of 3.4 feet. Total precipitation recorded at Boston Logan Airport was 2.6 inches, preceding the peak tide for more than 12 hours. The October 1991 northeaster or "Halloween Storm" was an extratropical event, which originally developed east of Nova Scotia, strengthening as it moved westward, and causing considerable damage along the eastern coast of the United States. Recorded tide levels at Boston were about 1 foot less than peak levels associated with the "Blizzard of 1978." Recorded peak wind gusts ranged from 78 mph at the National Weather Service Office in Chatham, 68 mph at Marblehead, and 55 mph at Boston. Rainfall data recorded at the Boston Logan Airport gage was also analyzed to address flooding. Although total recorded rainfall for this period was 2.71 inches the peak rainfall occurred more than 24 hours after peak tide; therefore, having little influence on flooding conditions for this storm. Comparative data for recent events, together with recorded ocean levels, windspeeds, and directions in the Nahant area, are presented in table B-13.

d. Hydrologic Zones. Based on collected information, the study area was subdivided into two hydrologic zones. Zone 1, located along Lynn Harbor, is directly affected by wave overtopping, no accumulation of floodwaters occurs in this zone. Zone 2 acts mostly as a reservoir, collecting all floodwaters from zone 1 and overtopping from Short Beach for rare events. Rainfall analysis of the zones established that rainfall contribution to local flooding was minimal, and that the largest percentage of flooding in both areas resulted from wave overtopping. A short description of the zones follows. Hydrologic zones and reaches and other relevant information are shown on plate B-2.

(1) **Zone 1**, is a small strip of land along Lynn Harbor, extending 200 feet south of the intersection of Castle Road and Castle Way to the intersection of Ward and Castle Roads. The width of the zone was estimated from the private seawalls to the opposite side of Castle Road. Street elevations at Castle Road range between 7.6 and 9.96 feet NGVD. Development of discharge rating curves and analysis of mapping indicated that as depth increases, a significant hydraulic gradient results, causing floodwater not intercepted by local drainage to discharge through the streets and convey in the Lowlands Park (zone 2). As described in section 6a, three reaches in the Lynn Harbor side contribute to flooding of zone 1. Reach 1 is open to the harbor and has ground level elevations lower than the 50 year event stillwater elevation. Varying elevations in reaches 2 and 3 offer partial protection against overtopping.

(2) The Lowlands Park and streets adjacent to zone 1 were identified as **Zone 2**. For hydrologic analysis, zone 2 was limited by Flash and Spring Roads to the south, and Nahant Road to the east. The Lowlands Park acts as storage basin for zones 1 and 2. Flooding in zone 2 is caused by floodwaters discharged through the streets from

TABLE B-13

RECENT FLOODS AT NAHANT, MASSACHUSETTS
COMPARATIVE HYDROLOGIC DATA

Flood Event	7 Feb 1978	19 Feb 1972	12 Nov 1968	25 Jan 1979	30 Oct 1991	12 Dec 1992
Ocean Tide (ft, NGVD) Observed*	10.4	9.2	7.8	9.3	9.4	9.4
Tide Frequency, Estimated (%)	0.9	10	100	8	7	7
Maximum 1-Hr. Rainfall (in.)	0.2	0.5	0.3	0.3	0.3	0.5
Storm Rainfall (in.)	2.8/48 hr	2.5/24 hr	1.8/24 hr	2.1/24 hr	2.5/48 hr	4.2/24 hr
Max. Wind (Fastest-mile, MPH)	44	47	54	45	37	51
Wind Direction	NE	NE	NE	E	NE	NE

* Observed values are as corrected by NOS, 1992

zone 1, and in less frequent events, by overtopping from Short Beach. Short Beach has revetment protection ranging in elevations of 13.0 and 14.0 feet NGVD. It was estimated that overtopping in this reach occurred over an 800-foot long section of the beach for 100-year events or greater. Floodwaters from this side of the study area discharge directly into Lowlands Park.

e. Interior Storage Capacity. An interior storage capacity curve for zone 2 was developed to determine overtopping volumes at reported elevations. Curves were estimated from 10-foot contour mapping, and other limited information such as spot elevations and historic high watermarks. Total capacity of the Lowlands Park area from estimated 2.0 feet NGVD low elevation to street level (approximately 4.3 feet NGVD) was approximated to 9 acre-feet, or an equivalent 1.7 inches of rainfall runoff. At this elevation, the water level inundates the park but does not overflow through adjacent streets.

f. Rating Curves. Most of the flooding into zone 1 occurs through reach 1. Stage-discharge relationships for reach 1 were estimated, using the weir equation with discharge coefficients ranging between 2.5 and 3.0. Overtopping volumes for reaches 2 and 3 were computed, based on peak rates of overtopping provided by EEHB, and added to obtain peak volumes for several flood frequencies.

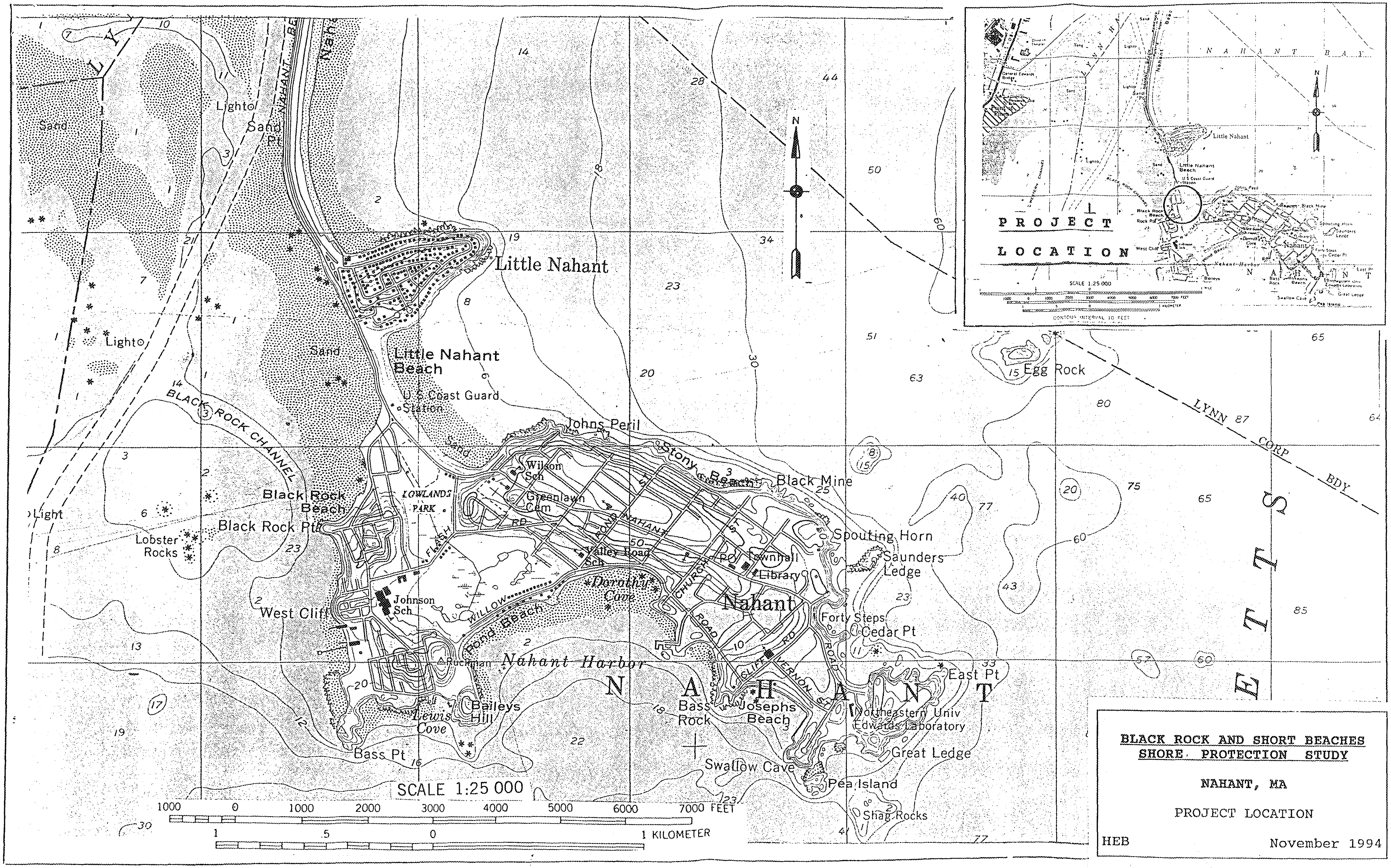
g. Existing Conditions Stage-Frequency Curve. Using interior storage capacity curves, discharge rating curves, and computed overtopping volumes, the stage-frequency curves, shown in plates B-3 and B-4, were developed for zones 1 and 2, respectively. Stages were then compared to information on historic flood levels collected from local residents and supplemented with surveyed high water marks to adjust and assume reasonableness of the curves.

(1) Zone 1. Flooding is caused directly by high stillwater levels flowing through reach 1 (Town Beach and houses with no flood protection) and to a lesser extent due to wave overtopping from reaches 2 and 3. There is no high watermark information in zone 1 for the 6-7 February 1978 storm. As mentioned earlier, this storm is estimated to have a 1 percent chance recurrence. Due to its proximity to the ocean, flood elevations for the 100-year event in zone 1 was estimated to reach 10.3 feet NGVD (ocean stillwater level). Some high watermark information for the October 1991 storm is available in zone 1. According to residents' accounts, and photographs taken by Corps personnel immediately after the storm, water levels reached elevations between 8.0 and 8.8 feet NGVD. This storm was assigned a frequency of

occurrence between 5 and 10 percent, based on the stillwater tide level.

(2) Zone 2. High water information for the February 1978 and October 1991 storms were available in zone 2. According to witness' reports and one resident's photographs, floodwaters reached elevations ranging between 9.3 and 10.2 feet NGVD during the February 1978 storm. The October 1991 storm was estimated to reach 6.9 to 7.6 feet NGVD water levels.

h. Modified Conditions. A 1,100-foot long seawall along reaches 1, 2, and 3 in zone 1 was determined as a possible alternative for this study. Two seawall elevations were analyzed, 12.5 and 13.0 feet NGVD. With a seawall at elevation 12.5, wave overtopping begins at a 100-year event. With this alternative, flooding is reduced to a few inches of water above the street. A seawall of elevation 13.0 eliminates overtopping up to and including a 100-year event. Analysis shows that zone 2 would still receive overtopping discharges from Short Beach, which may raise the level in Lowlands Park to approximately 5.2 feet NGVD which, represents approximately 1 foot above street level for the 100-year event. More frequent events would have the same effect in both zones, a few inches of water above street level caused mainly from rainfall runoff; however, they would not result in significant interior flood elevations.



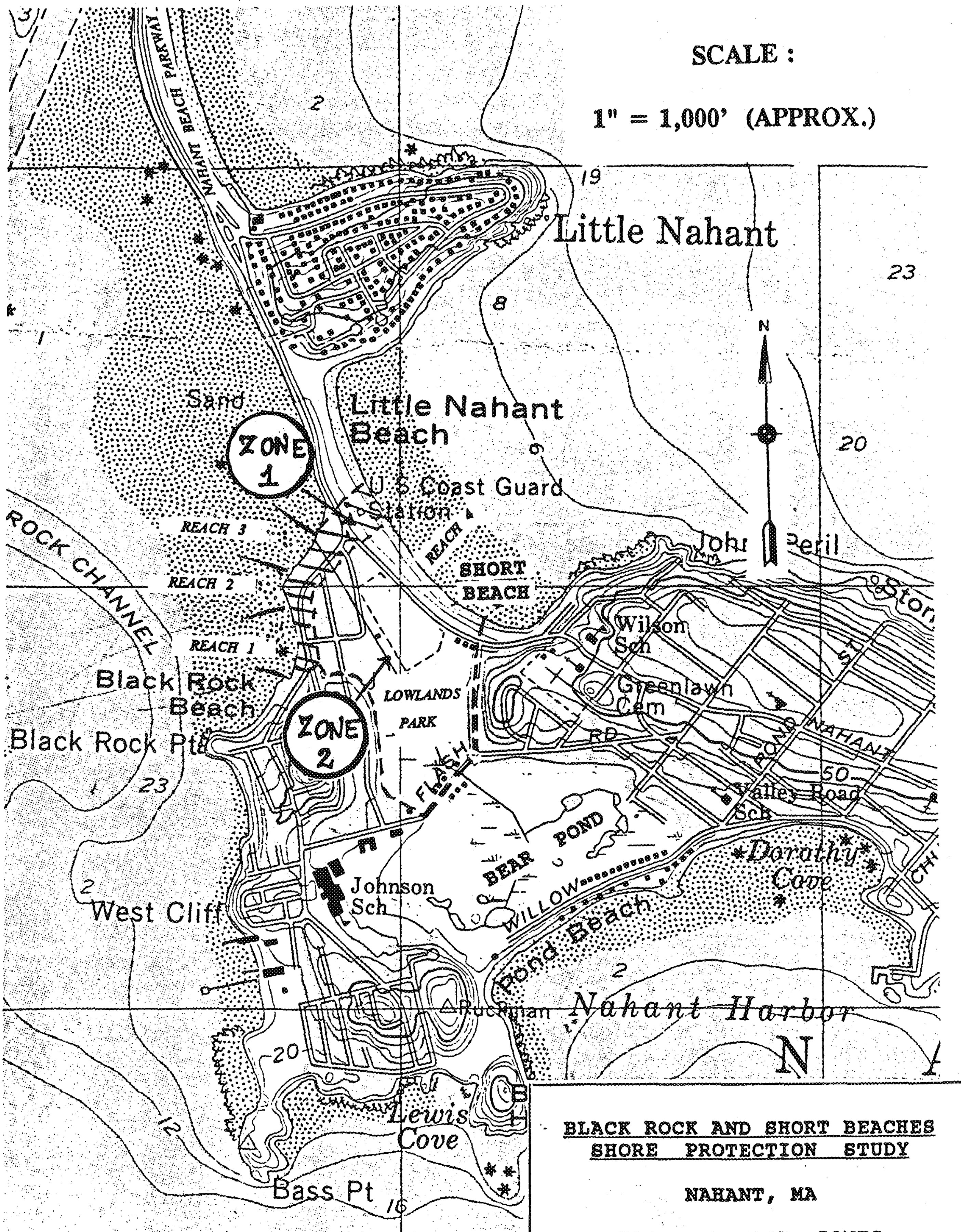
BLACK ROCK AND SHORT BEACHES SHORE PROTECTION STUDY

NAHANT, MA

PROJECT LOCATION

HEB November 1994

PLATE B-1



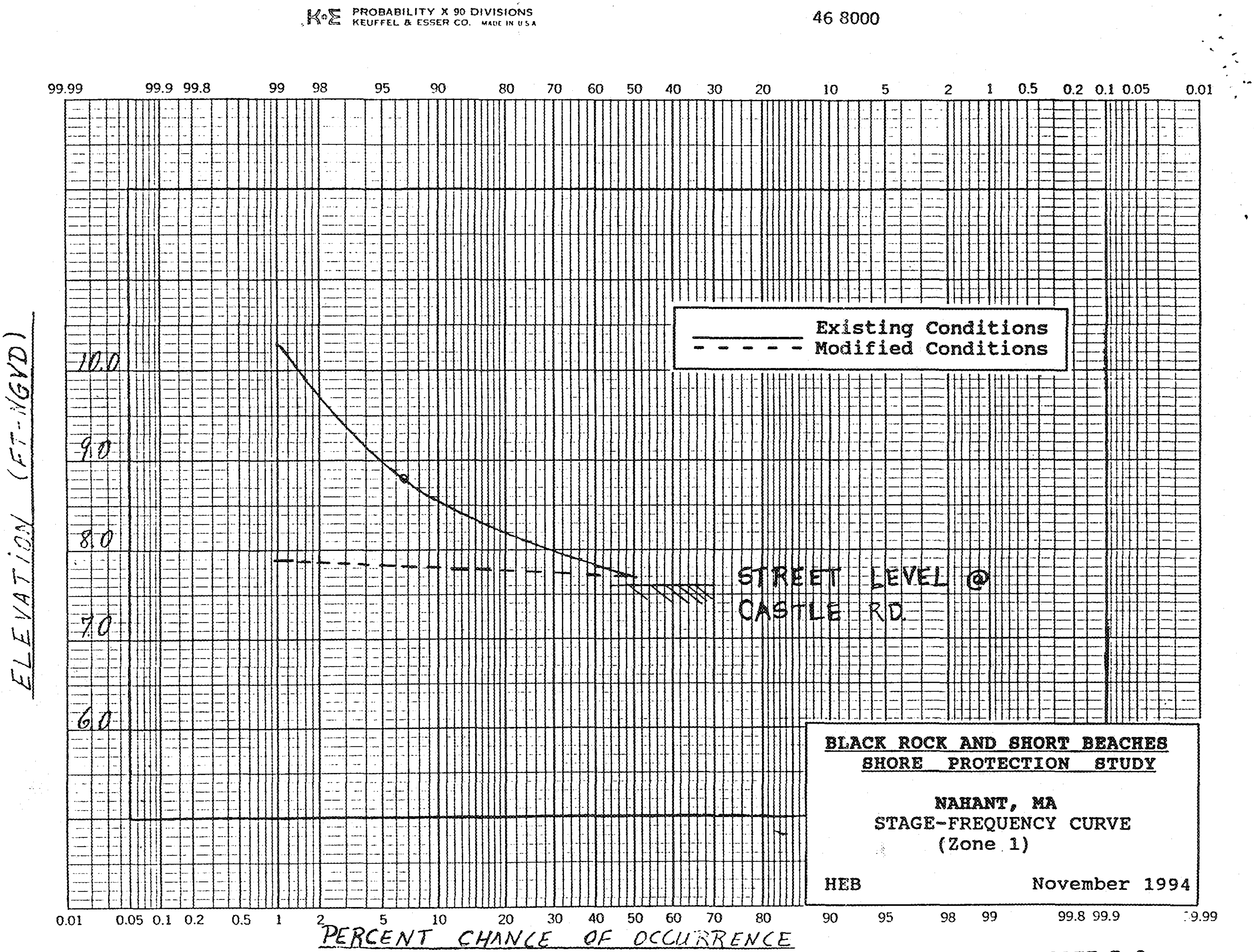
BLACK ROCK AND SHORT BEACHES SHORE PROTECTION STUDY

NAHANT, MA

FLOOD DAMAGE ZONES

HEB

November 1994



BLACK ROCK AND SHORT BEACHES
SHORE PROTECTION STUDY

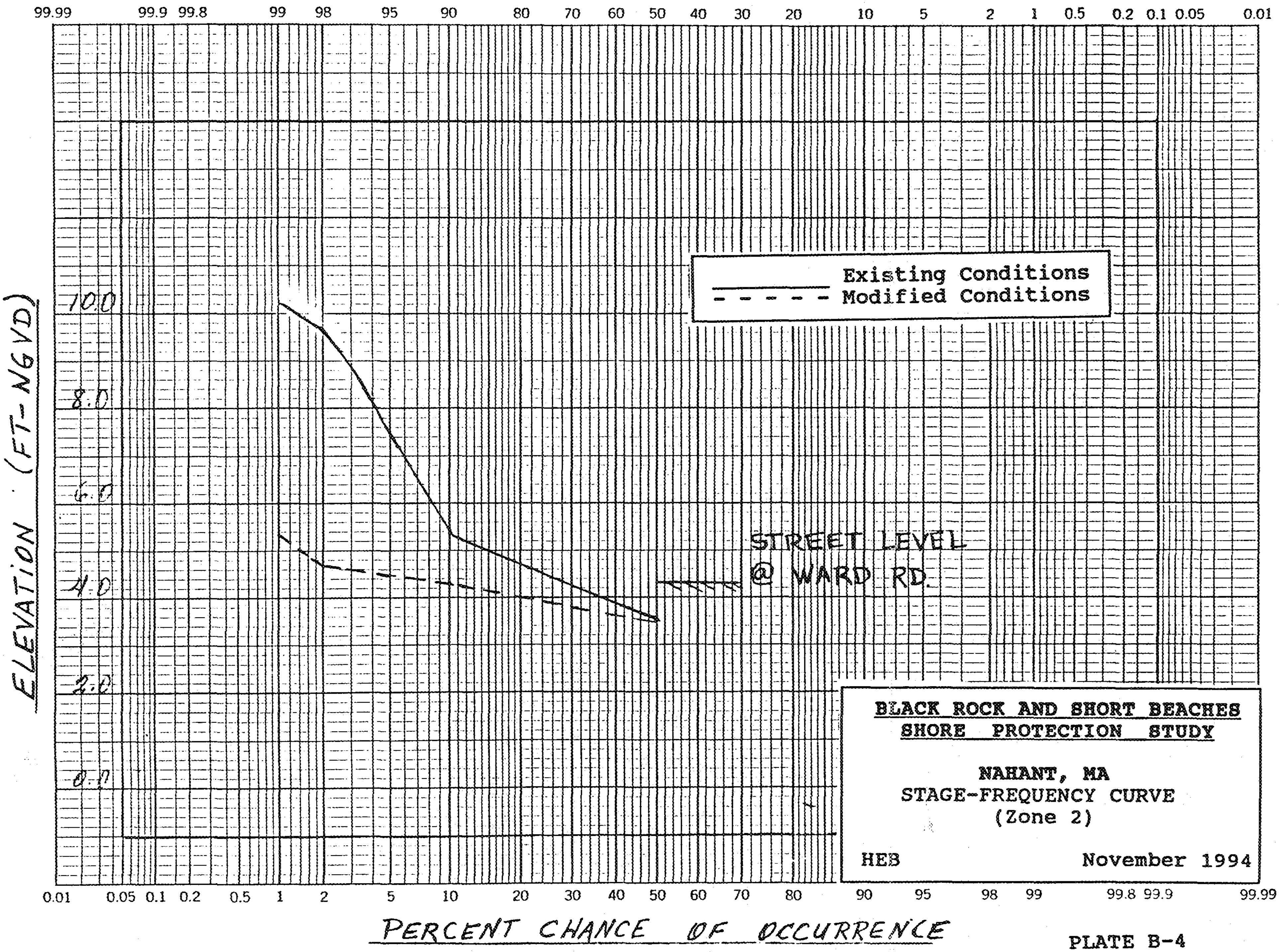
NAHANT, MA
STAGE-FREQUENCY CURVE
(Zone 1)

HEB November 1994

PLATE B-3

K+E PROBABILITY X 90 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 8000



BLACK ROCK AND SHORT BEACHES
SHORE PROTECTION STUDY

NAHANT, MA
STAGE-FREQUENCY CURVE
(Zone 2)

HEB **November 1994**

PLATE B-4

APPENDIX C
GEOTECHNICAL ENGINEERING

SECTION 103 RECONNAISSANCE STUDY
NAHANT SHORE PROTECTION STUDY
NAHANT, MASSACHUSETTS
GEOTECHNICAL APPENDIX

SECTION 103 RECONNAISSANCE STUDY
NAHANT SHORE PROTECTION STUDY
NAHANT, MASSACHUSETTS
GEOTECHNICAL APPENDIX

1. SUMMARY.

A Section 103 Reconnaissance Study of erosion and flood protection was conducted at Black Rock Beach and Short Beach in Nahant, Massachusetts. This study was conducted by the U.S. Army Corps of Engineers, New England Division. Three potential alternatives are proposed to provide protection to 1,000 feet along Black Rock Beach: (1) beach nourishment, (2) dike construction, and (3) reinforced concrete T-wall construction. The required length for the two latter alternatives is dependent upon the possibility of tying into the existing seawall. If it is possible to tie in, the required length would be approximately 430 feet. However, permits and capping of the existing walls having insufficient height would be necessary. If that is not determined to be feasible, an alternative method of new construction must be applied to the entire length of 1,100 feet. In either case, the protective structure would be constructed near the street for the 150 feet of public beach, and along the shoreline for the rest of the distance, 950 feet. To connect either of the latter two proposed structures, a tie-in must be constructed, connecting the seaward structure with the wall near the street, a length of approximately 45 feet. Local materials are available for construction. Available data for Short Beach indicates that no further analysis is required for proposed alternative protection based on the closeness of the design and existing elevations.
The site geological setting, natural construction materials proposed for use (sand, gravel and stone), material availability information, design and construction considerations for the proposed alternatives follow.

2. TOPOGRAPHY.

2.1 General. Nahant is located in the Lynn, Massachusetts Quadrangle. The study area lies within the Coastal Lowlands, a broad northeast-trending belt from south of the Rhode Island coast north to Augusta, Maine. While areas of the Lowlands attain elevations of up to 360 feet above mean sea level, the greater part of the land along the coast is less than 60 feet above mean sea level. The majority of the immediate project area is less than 50 feet above mean sea level. Local relief consists of a relatively flat area, an area of depression which is occasionally flooded, beach deposits, bedrock outcrops, and the connecting causeway between Nahant/Little Nahant and Lynn. Topography in the study area has been influenced by four factors: presence of bedrock near the surface, the effects of glaciation, coastal processes, construction and alteration by man.

2.2 Black Rock Beach. Black Rock Beach is located in the Lynn, Massachusetts quadrangle, just east of the narrow stretch of land leading into Nahant along Nahant Road (Figure 1). It is located on the south side of the stretch bounded by Lynn Harbor, and is approximately 1,100 feet long. The approximate slope of the beach is 1 vertical on 10 horizontal with an average crest elevation of 9.8 feet NGVD. There is a lowlands area behind Black Rock Beach with elevations ranging from 4 feet to 10 feet NGVD.

Development along Black Rock Beach consists of private homes and a public beach area which is approximately 150 feet long. Approximately 65%, or 865 feet of the beach currently has some form of seawall, although the existing height may not be adequate to withstand a 50- or 100-year storm.

2.3 Short Beach. Short Beach is located on the east side of the causeway between Little Nahant and Nahant (Figure 1). There are no private dwellings abutting the beach. Figure 5 indicates that the existing crest height at Short Beach (13.5 feet NGVD) is above the required crest height for a 50-year storm (13.2 feet NGVD) and 0.4 feet below the required height for a 100-year storm (13.9 feet NGVD). Since the design and existing elevations are fairly close, and none of the alternatives are applicable to Short Beach, there is no basis for further study. Therefore, no further analysis of flood protection shall be conducted for Short Beach. Figure 5 data supports local citizen input that overtopping does not occurr until they experience storms having 50- to 100-year return periods.

3. GEOLOGY.

3.1 General. The geology of New England is the result of a complicated history of orogeny, intrusion, and metamorphism. There are mixed rock types in very complex associations. Although numerous faults have been mapped or otherwise suspected, none are presently known to be active. The area has been glaciated several times and the modern landscape is largely one of remnant surficial deposits of glacial origin overlying bedrock. Six different rock types are shown on the bedrock geology map of Nahant.

3.2 Bedrock Geology. Nahant is located approximately 9 miles northeast of Boston, MA. Much of the island contains coastal bedrock exposures and cliffs up to approximately 50 feet high. Most of Nahant is underlain by the Nahant gabbro. Based on data obtained from the Soil Survey of Essex County, Massachusetts, it is possible that the depth to bedrock in some places under the project alignment area may be as much as 2 feet. Bedrock outcrops were observed on the Island but none were observed within the study area. A preliminary bedrock map of the Nahant area is available, see Figure 2 for additional information.

3.3 Surficial Geology. Sediments overlying the bedrock are variable ice contact materials (till), stratified drift (sand and gravel, with minor silt, clay, and till), and recent beach deposits. Glacial till overlies the bedrock throughout most of the study area. Till is overlain in places by stratified drift, deposited by glacial meltwaters during retreat of the ice sheets. The Hollis soils within the study area are described as "moderately deep or shallow, gently sloping to steep, well drained or somewhat excessively drained, loamy soils formed in glacial till with areas of exposed bedrock" (Figure 3). The beach materials at both Short Beach and Black Rock Beach are primarily a quartz-rich fine grained sand.

4. GENERALIZED SUBSURFACE CONDITIONS.

Soil explorations and laboratory testing were not performed as they are not within the scope of this reconnaissance study. There was no subsurface information readily available from other projects in the study area at this time. Therefore, subsurface conditions were evaluated by reviewing geologic literature and making interpretations based on observations made during the site visit.

The study area includes three types of materials: beach sand, drained Hollis soils, and bedrock. The beach material primarily contains quartz sand with minor amounts of accessory minerals, along with occasional gravel and cobblestones. The drained Hollis soils contain fine sandy loam, with a moderate to moderately high permeability rate. On the surface, the soil in this project alignment consists of light brown fine grained, uniformly graded sand. The typical soil profile within the study area appears to be granular beach deposits underlain by glacial till deposits and bedrock which may be at a shallow depth of less than 2 feet.

5. MATERIAL AVAILABILITY.

5.1 General. The sand specifications used were for a sand with 100 % passing a 3/8 inch screen and less than 5 % passing through a #200 screen which matches a concrete sand. Pricing data were obtained from area producers for a processed concrete sand. The existing sand at Black Rock Beach is primarily a quartz rich fine-grained sand. A materials survey was conducted using available file information and calling of sand suppliers. Offshore sand sources were not examined as part of this reconnaissance study.

5.2 Beach Fill. A preliminary material availability survey was conducted of land based sand sources for Black Rock Beach. The sand quantity of 12,000 cubic yards needed for this project was based on estimates for a 100 year storm event. A total of nine sand suppliers were contacted for this reconnaissance study. Three suppliers could not handle the quantity required. One supplier said that he was "too far" from the project location and one did not call back with a price quote. Prices were requested for processed concrete sand and ranged in price from $7.25 per ton delivered to $12.60 per ton delivered. The lowest price of $7.25 per ton delivered was from a sand supplier located approximately 50 miles south of the site. The closest supplier is located approximately 10 miles southwest of the site and could deliver the sand at a price between $9.00 per ton and $9.50 per ton. Concrete sand was considered suitable for this project.

5.3 Dike Materials.

5.3.1 General. The materials required for the dike construction, in addition to beach fill are compacted impervious fill, gravel filter and cover stone layer. The grain size distribution graph for the fill, gravel filter and stone cover layer is shown on Figure 4. If the dike is to be tied into existing structures, the required length would be greater than the 430 feet of the beach front length without structures, since there would have to be a significant overlap between the proposed and existing structures. A more realistic estimate is 595 linear feet required for construction of the dike. File information indicates that there are sufficient stone suppliers within a 50 mile radius of the study area to satisfy the quantities required for the dike materials below.

5.3.2 Compacted Impervious Fill. Compacted Impervious Fill is required for the core of the dike. For this study, the estimated volume needed for the impervious fill is 2,070 cubic yards required for tie-ins and 4,000 cubic yards to transcend any existing structures.

5.3.3 Gravel Filter. The material required is a poorly graded gravel with sand. The quantity of gravel needed for the project is 925 cubic yards for tie-ins and 1,785 cubic yards for the proposed dike.

5.3.4 Cover Stone Layer. The material required is a crushed stone, approximately 1 to 10 inches in diameter, with a weight ranging from 1.5 to 50 pounds. The quantity of stone required is 1,485 cubic yards for the tie-in option and 2,862 cubic yards for the independent dike. No suppliers were called for price quotes, but prices from a recent similar study in the area were used for estimating purposes. Stone prices vary from $8 per ton delivered to $17.30 per ton delivered for similar material.

5.4 Reinforced Concrete T-Wall.

The materials required for construction of a reinforced concrete T-Wall include concrete, 110 p.c.f. reinforcing steel, temporary steel sheet piles, rock toe material and gravel bedding. The gravel bedding and rock toe material fit into the same category as the gravel filter and cover stone layer, as discussed in Section 5.2.3 and 5.2.4 above, respectively. Approximately 390 cubic yards are necessary for the gravel bedding layer tie-in option and 890 cubic yards for building in front of the existing structures. The total rock fill volume needed is 630 cubic yards for the tie-in option and 1,450 cubic yards for the entire structure. Concrete quantity for the tie-in is 520 cubic yards. Total quantity of concrete required is 1,350 cubic yards.

6. DESIGN AND CONSTRUCTION CONSIDERATIONS.

6.1 General. Design and construction considerations are provided in the following sections for the dike and T-Wall proposed alternatives. The design was completed assuming that the structure would be constructed near the street for the 150 feet of public beach, and along the shoreline for the rest of the distance, 950 feet, with a tie-in connecting the proposed structure near the shoreline and the proposed structure near the street.

6.2 Dike. A preliminary dike section was developed using the principles and procedures outlined in the "Shore Protection Manual". This section is shown on Plate 1. This section was used in calculating the quantities needed to prepare the estimates for the dike materials in Section 5. Design wave heights and still water levels were provided by New England Division's Coastal Engineering Branch of Engineering Division and are discussed more thoroughly in the 30 August 1994 report. Preliminary design of the dike assumes there would be a fronting beach maintained at elevation 3.0 feet NGVD and no overtopping. Top elevation of the impervious fill was determined by adding the height of the still water level (10.3 feet NGVD) and the runup of the design wave during a 100-year storm (1.6 feet).

Formal stability, settlement and seepage analyses were not performed on the sections because there are no exploration or laboratory test data available to justify performing the analyses at this time. Based on NED experience with similar materials, the proposed 1 vertical to 1.5 horizontal slopes are judged to be stable. The proposed gravel filter was designed to prevent migration of fines.

6.3 Reinforced Concrete T-Wall. The reinforced concrete T-Wall was based on EM 1110-2-2502 Retaining and Flood Walls. A typical section for a 100-year storm is shown on Plate 2. The width required for this alternative is approximately 25 feet, whereas the dike would occupy as much as 45 feet. Preliminary design of the T-Wall was done assuming that there would be a fronting beach maintained at 3 feet NGVD. Detailed design analysis for the T-Wall will be performed in the feasibility stage if further study is warranted.

6.4 Constructibility. Access to Black Rock Beach may be obtained by removing part of the wall at the top of the public beach area. The need to raze any existing flood control structures is dependent upon the dimensions, locations and conditions of the existing structures. It may be possible to build the dike over certain existing structures. For construction of the T-Wall, sheetpiling and dewatering are necessary, due to the closeness of the shoreline. Also, borings should be taken during the feasibility study to determine the depth to bedrock to better evaluate the alternative of choosing the T-Wall. A survey should be conducted of the existing structures if the tie in option is chosen to determine if additional capping is necessary.

7. RECOMMENDED FUTURE STUDIES.

Explorations (borings and/or test pits) should be conducted as part of the feasibility study to facilitate the design of any dike or wall warranting further study. Samples of the existing beach materials should be collected and tested (grain size analysis) to facilitate the development of a sand specification for beachfill alternatives warranting further study.

REFERENCES

Bell, K.G., 1977, Preliminary Bedrock Geologic Maps of the Lynn and Marblehead South Quadrangles, Massachusetts. U.S. Geological Survey Open File Report 77-180.

Bailey, R.H.; Ross, M.E., 1993, Geology of East Point, Nahant, Massachusetts - Chapter Y. Field Trip Guidebook for the Northeastern United States: 1993 Boston GSA, Volume 2. Geological Society of America Annual Meeting, Boston, Ma, October 25 - 28, 1993.

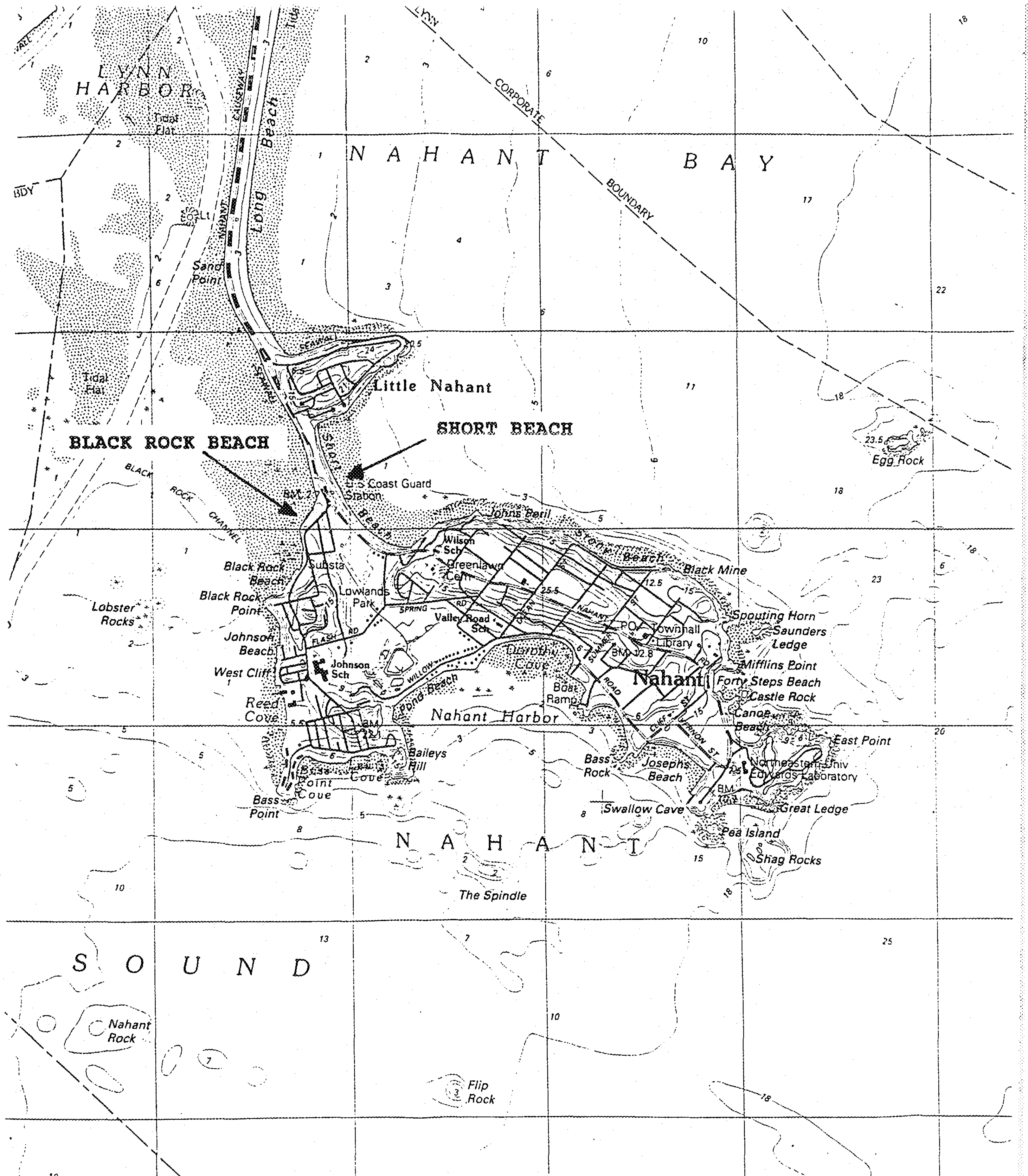
Denny, C.S., 1982, Geomorphology of New England. U.S. Geological Survey Professional Paper 1208, 18 p.

U.S. Army Corps of Engineers, Engineering Manual 1110-2-2502, Retaining and Flood Walls, 29 September 1989.

U.S. Army Corps of Engineers, Shore Protection Manual, 1984, Coastal Engineering Research Center, Waterway Experimentation Station, Vicksburg, Mississippi.

U.S. Department of Agriculture, Soil Conservation Service, Soil Survey of Essex County, Massachusetts, Southern Part, 1984.

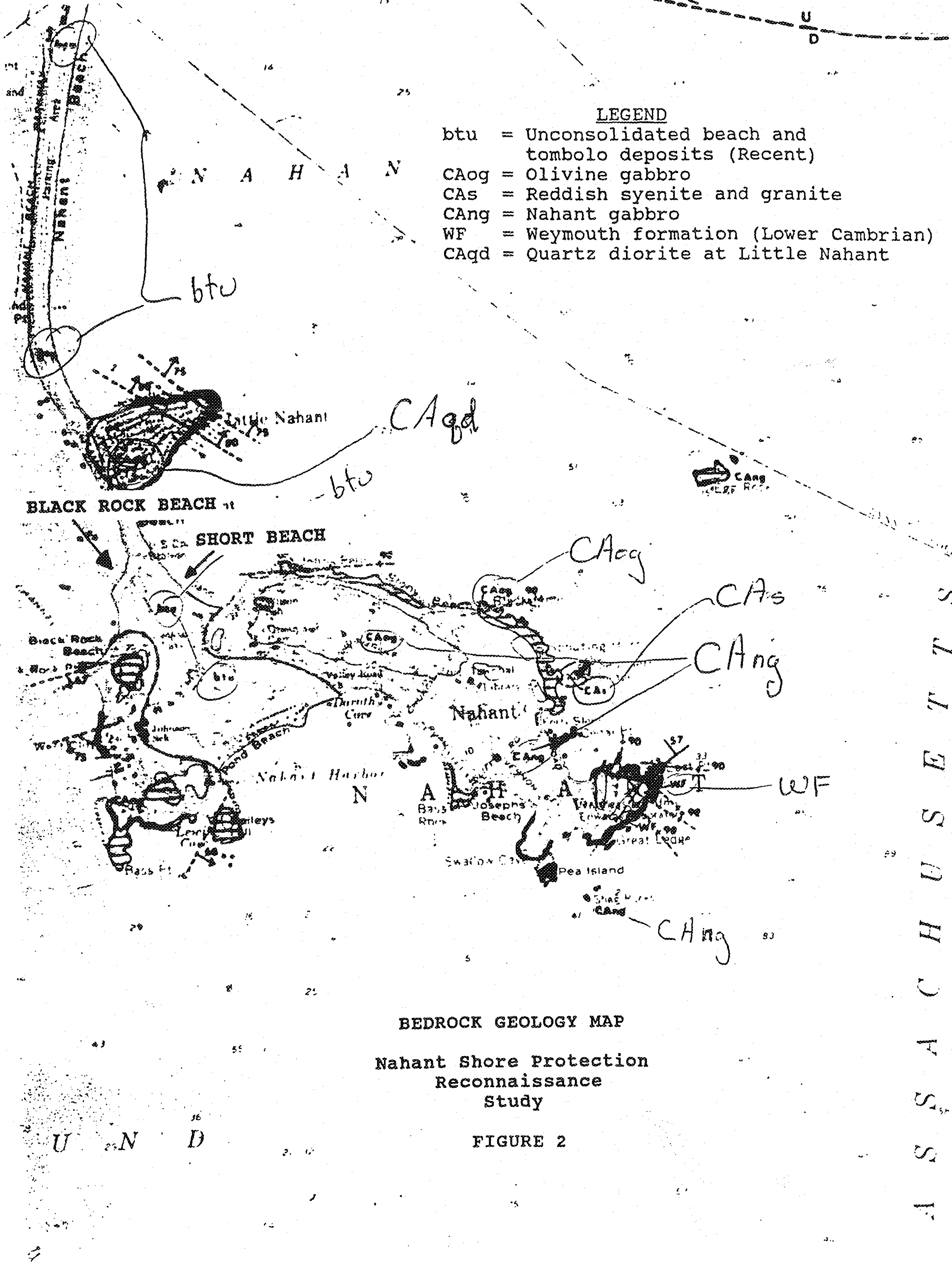
U.S.G.S. 1985. Lynn, Massachusetts Quadrangle. U.S. Geological Survey 7.5 x 15 Minute Series, Topographic.



LOCATION MAP

Nahant Shore Protection
Reconnaisance
Study

FIGURE 1



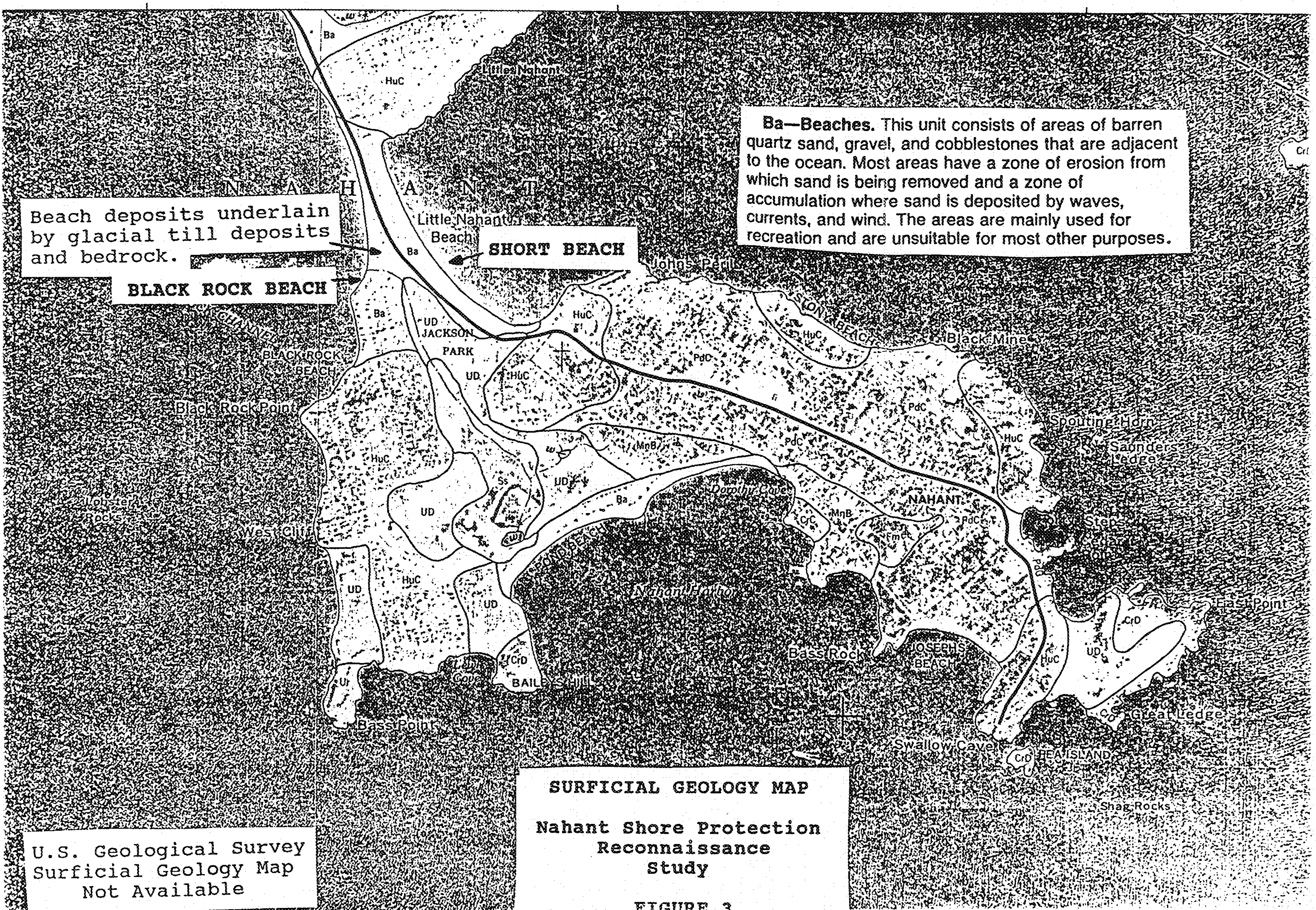
LEGEND

btu = Unconsolidated beach and tombolo deposits (Recent)
CAog = Olivine gabbro
CAs = Reddish syenite and granite
CAng = Nahant gabbro
WF = Weymouth formation (Lower Cambrian)
CAqd = Quartz diorite at Little Nahant

BEDROCK GEOLOGY MAP

Nahant Shore Protection Reconnaissance Study

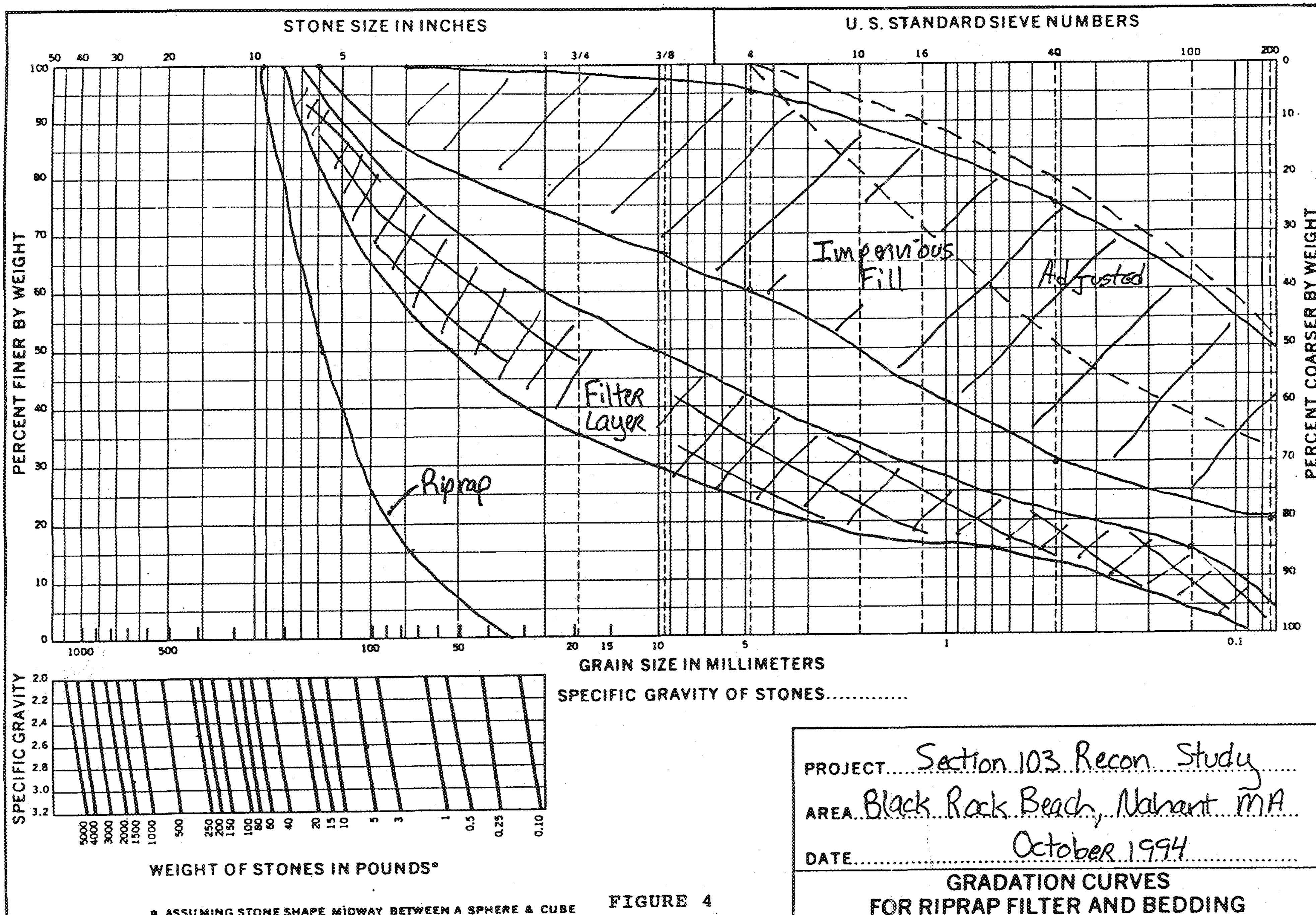
FIGURE 2



SURFICIAL GEOLOGY MAP

Nahant Shore Protection Reconnaissance Study

FIGURE 3



SPECIFIC GRAVITY OF STONES............

FIGURE 4

PROJECT Section 103 Recon Study

AREA Black Rock Beach, Nahant MA

DATE October 1994

GRADATION CURVES FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056
APR 67

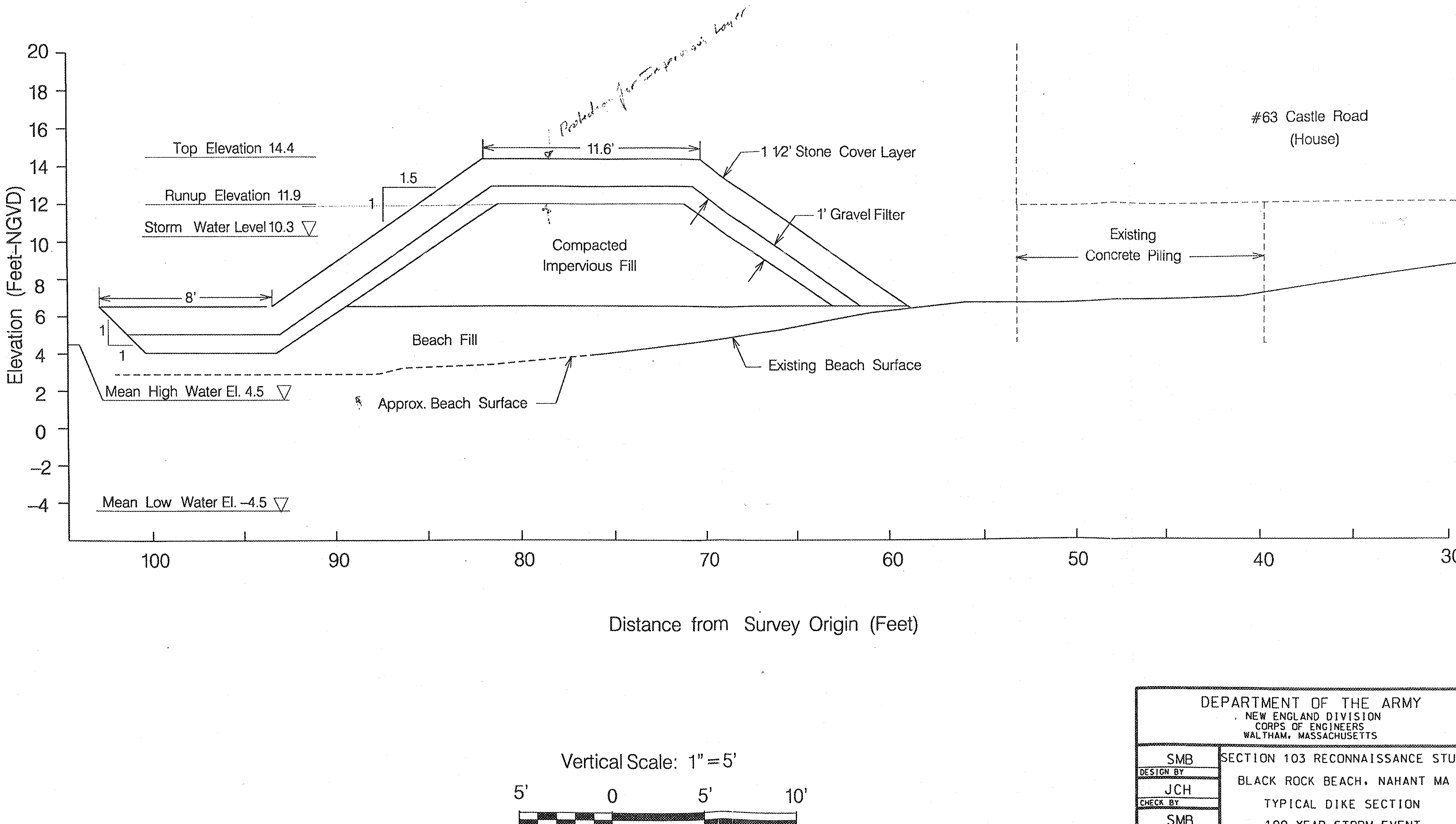
SHORT BEACH CREST HEIGHT

**Nahant Shore Protection
Reconnaissance
Study**

FIGURE 5

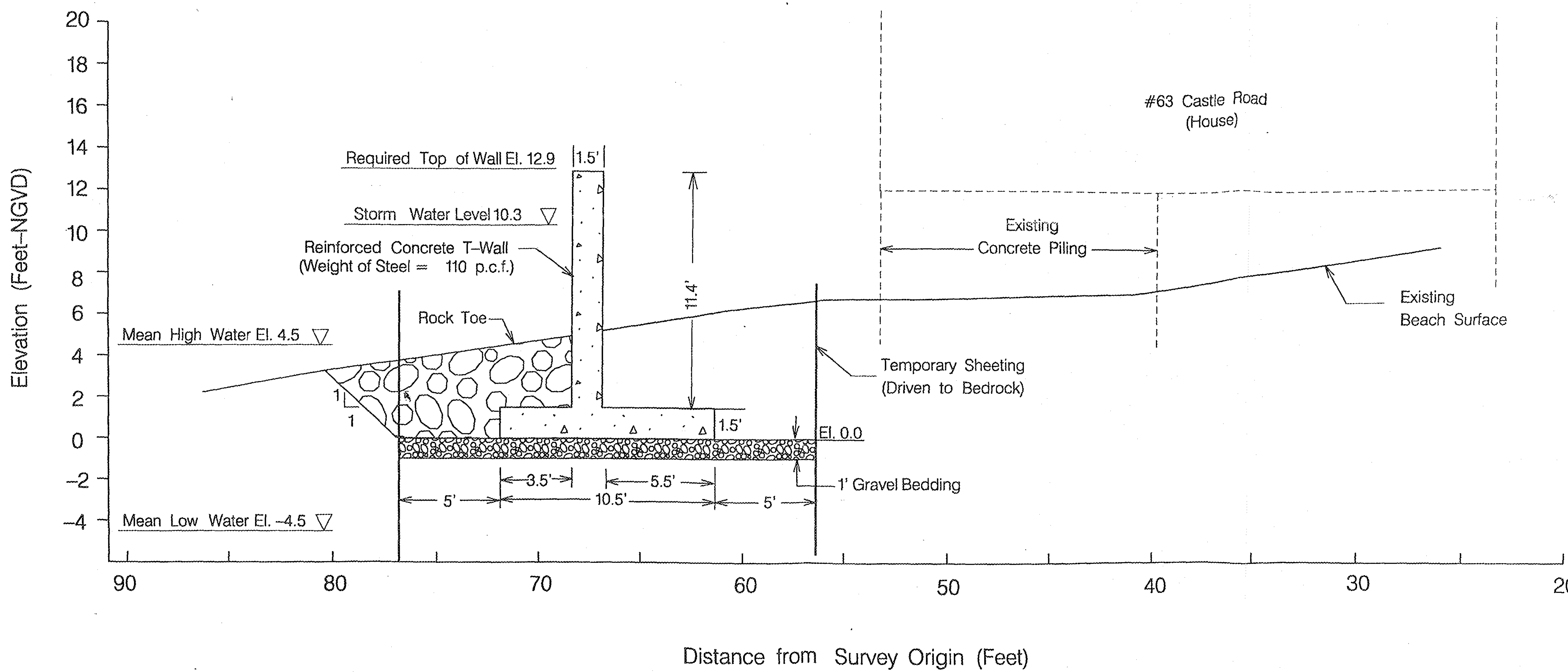
Event	SWL (NGVD)	Runup (ft)	Required Crest (NGVD)	Existing Crest (NGVD)
2	8.2	1.7	9.9	13.5
5	8.7	2.1	10.8	13.5
10	9.1	2.5	11.6	13.5
25	9.6	2.8	12.4	13.5
50	10.0	3.2	13.2	13.5
100	10.3	3.6	13.9	13.5

Existing crest is average of 3 profiles
(13.04 + 14.04 + 13.47 = 40.55 / 3 = 13.5)

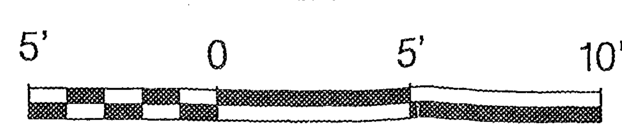


DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

SMB DESIGN BY	SECTION 103 RECONNAISSANCE STUDY
JCH CHECK BY	BLACK ROCK BEACH, NAHANT MA
SMB DRAWN BY	TYPICAL DIKE SECTION 100-YEAR STORM EVENT
GEOTECH. ENG. DIV. PLATE NO. 1	SCALE: 1" = 5' DATE: NOV. 1994



Vertical Scale: 1" = 5'



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

SMB/DD DESIGN BY	SECTION 103 RECONNAISSANCE STUDY
JCH CHECK BY	BLACK ROCK BEACH, NAHANT MA
SMB DRAWN BY	TYPICAL T-WALL SECTION 100-YEAR STORM EVENT
GEOTECH. ENG. DIV.	SCALE: 1" = 5'

TABLE 1

SUMMARY OF QUANTITIES

	Rock Required (c.y.)	Gravel Required (c.y.)	Impervious Fill Req'd. (c.y.)	Concrete Required (c.y.)
Tie-in Option Dike	1,800	1,115	2,450	N/A
Complete Dike	2,862	1,785	4,000	N/A
Tie-in Option T-Wall	915	550	N/A	720
Complete T-Wall	1,450	890	N/A	1,350